Page

CHAPTER 6 - STABILITY DESIGN

Page

Page

247-927 O - 78 - 10

Page

Page

CHAPTER 6. STABILITY EVALUATION AND DESIGN

Introduction

The analysis of earth channels with acceptable limits of stability is of primary importance to Soil Conservation Service activities. The evaluation or design of any water conveyance system that includes earth channels requires knowledge of the relationships between flowing water and the earth materials forming the boundary of the channel, as well as an understanding of the expected stream response when structures, lining, vegetation, or other features are imposed. These relationships may be the controlling factors in determining channel alignment, grade, dimensioning of cross section and selection of design features to assure the operational requirements of the system.

The methods included herein to evaluate channel stability against the flow forces are for bare earth. When evaluations indicate the ability of the soil is insufficient to resist or tolerate the forces applied by the flow under consideration it may be necessary to consider that the channel has mobile boundaries. The magnitude of the channel instability needs to be determined in order to evaluate whether or not vegetative practices and/or structural measures are needed. Where such practices or measures are required, methods of analysis that appropriately evaluate the stream's response should be used.

Figure 6-1 provides general guidance in selecting evaluation procedures that apply to various site conditions.

All terms used in this chapter are defined in the glossary on page 6-87.

Stability Evaluation

Methods presently used by the SCS in the evaluation of the stability of earth channels are based on the following fundamental physical concepts.

1. Essentially rigid boundaries. Stability is attained when the interaction between flow and the material forming the channel boundary is such that the soil boundary effectively resists the erosive efforts of the flow.

   Where properly evaluated and designed the bed and banks in this class of channels remains essentially unchanged during all stages of flow. The principles of hydraulics based on rigid boundaries are applicable in analyzing such channels.

   The procedures described in this chapter that are based on this definition of stability are:

   a. Allowable velocity approach.

   b. Tractive stress approach.

   c. Tractive power approach.

CHANNEL EVALUATION PROCEDURAL GUIDE

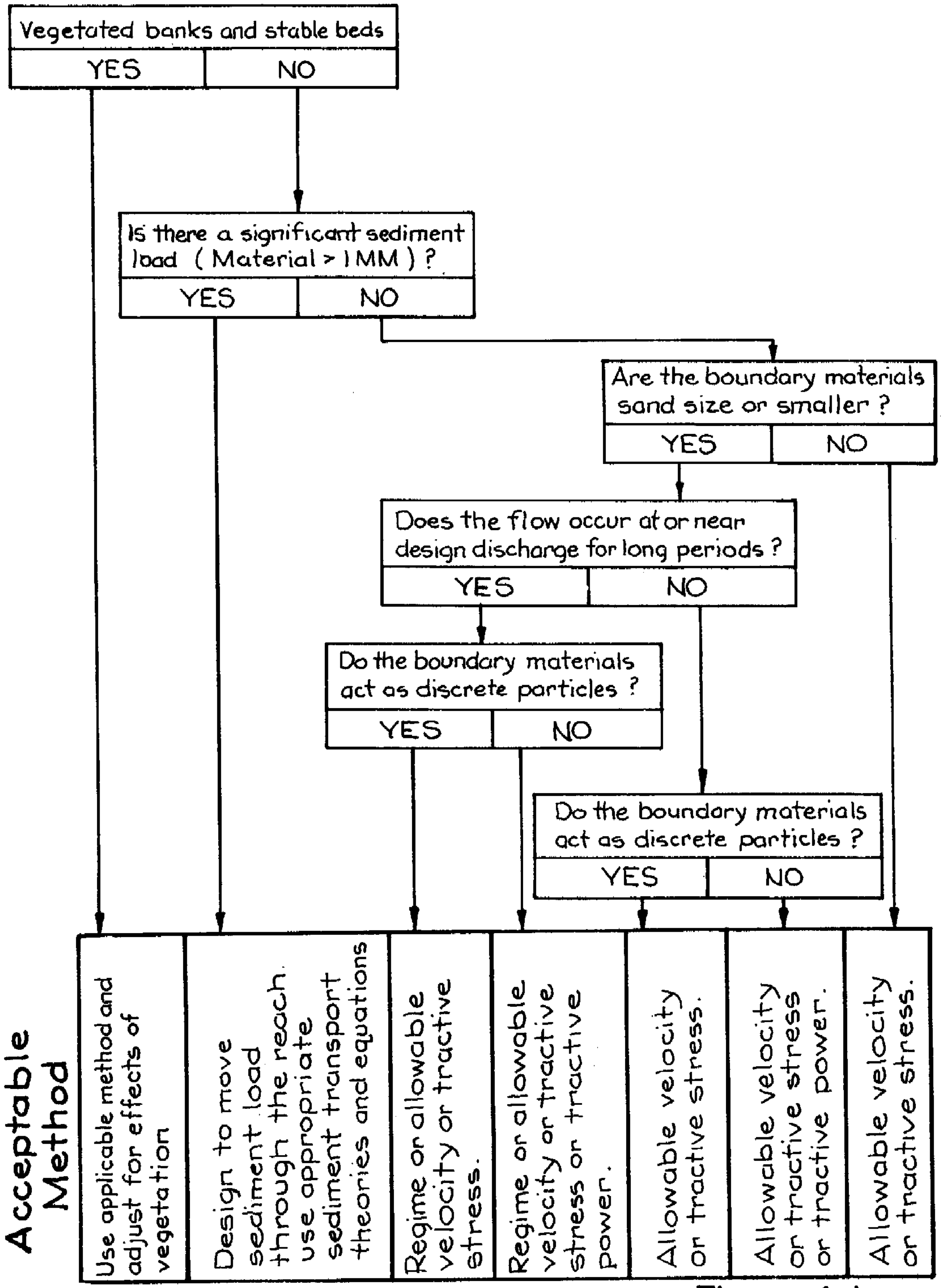


Figure 6-1

2. Mobile Boundaries. Stability is attained when the rate at which sediment enters the channel from upstream is equal to the capacity of the channel to carry material having the same composition as the incoming sediment. The bed and the banks of the channel are mobile and may vary somewhat from designed position. Stability in such channels may be determined by methods that use the principles of flow in channels with movable boundaries.

The procedures described in this chapter that are based on this definition of stability are:

a. Sediment Transport Approach.

b. Modified Regime Approach.

Procedure for Determining Sediment Concentration

The stability of a channel is influenced by the concentration and physical characteristics of the sediment entering the channel and available for transport as bedload and in suspension. Procedures for computing sediment transport are described in NEH-3, Chapter 4.[52] If clear water is not used, stream gage data when available and representing a wide range of flows are useful in predicting sediment loads. When the clear water procedure is not chosen and suitable data are not available, there is a method of making rough estimates of sediment loads presented in Geologic Note 2.

Allowable Velocity Approach

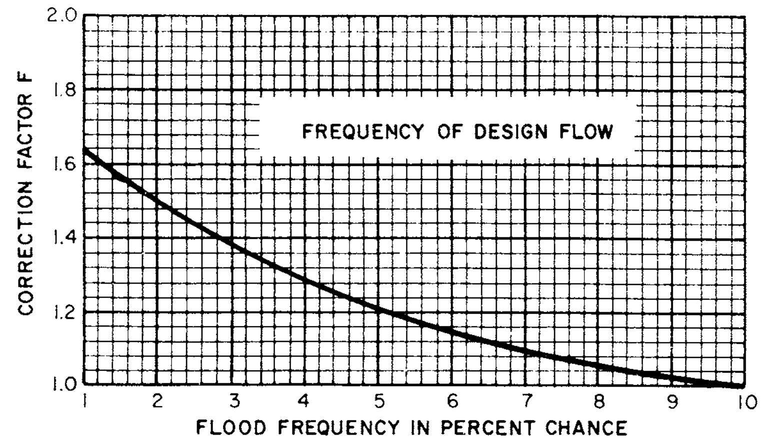
General

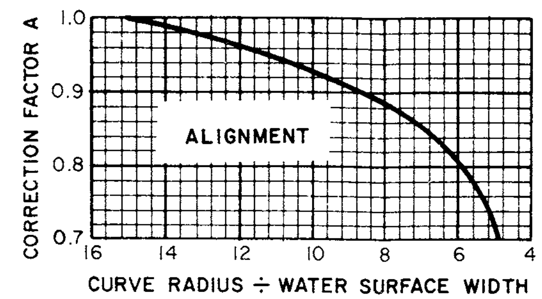
This method of testing the erosion resistance of earth channels is based on data collected by several investigators.

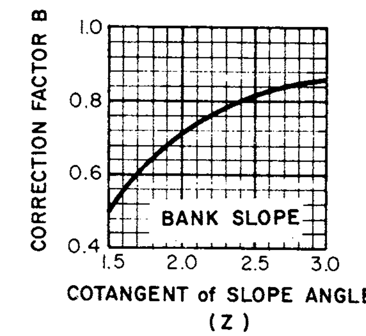
Figure 6-2 shows "Allowable Velocities for Unprotected Earth Channels" developed chiefly from data by Fortier and Scobey[24], Lane[25], by investigators in the U.S.S.R.[26] and others. The allowable velocities determined from Figure 6-2 refer to channels formed in earth with no vegetative or structural protection. The Fortier and Scobey data shown on Figure 6-2 were collected by the authors from engineers experienced in irrigation systems. The canals were well-seasoned, were on low gradients, and had flow depths of less than 3 feet.

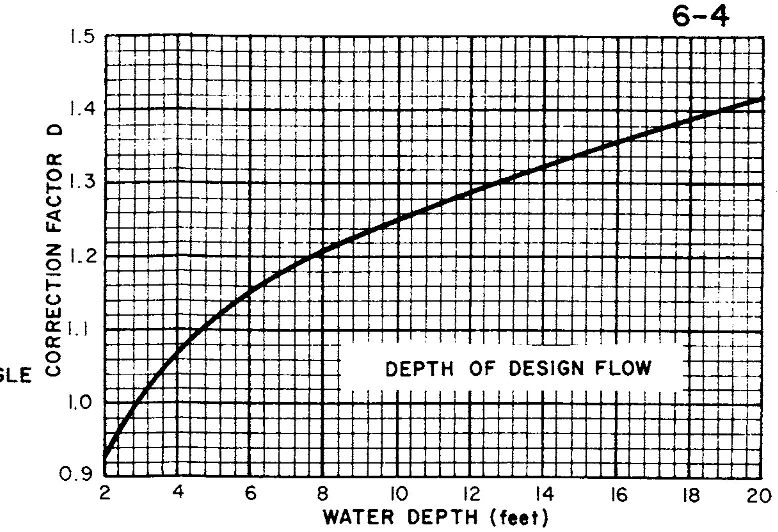
Stability is influenced by the concentration of fine material carried by the flow in suspension. There are two distinct types of flow depending on concentration of material in suspension.

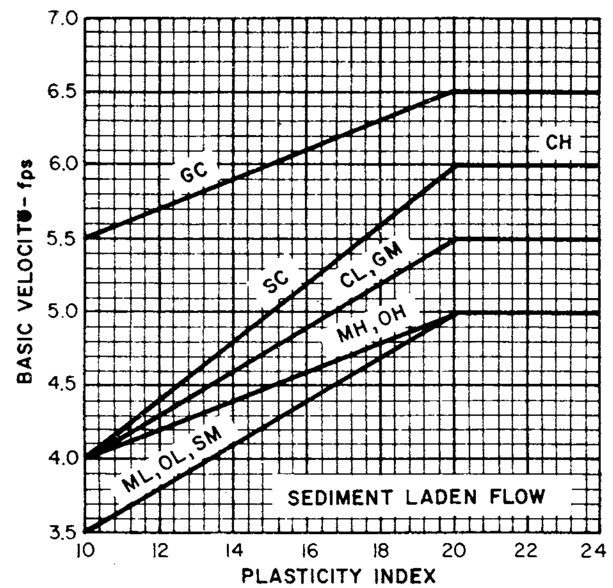
1. Sediment free flow is defined as the condition in which fine material is carried in suspension by the flow at concentrations so low that it



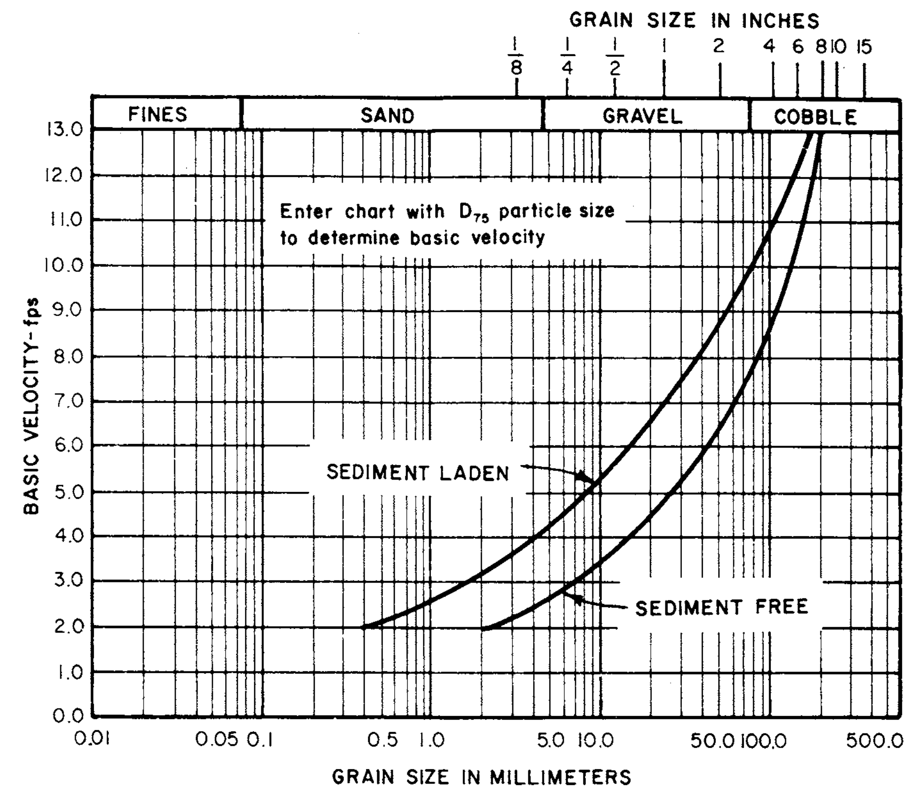




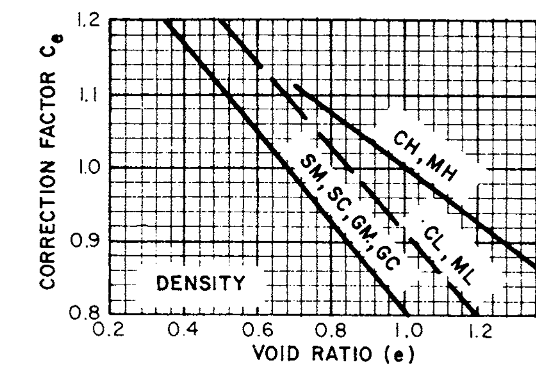


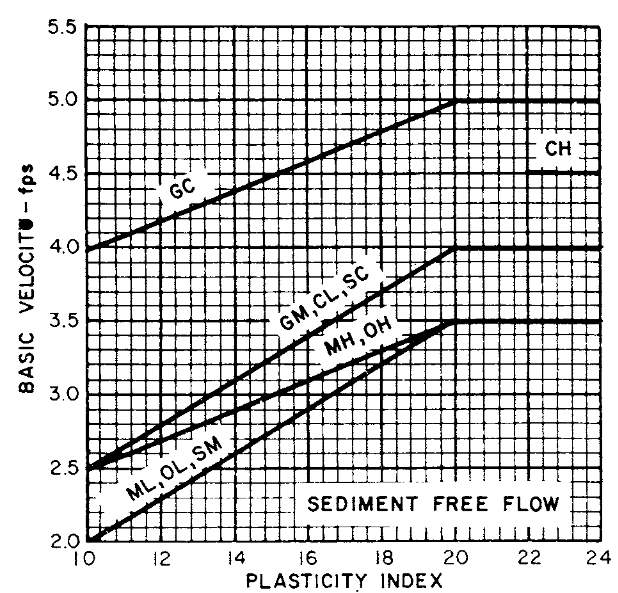


BASIC VELOCITIES FOR COHERENT EARTH MATERIALS (v_b)



BASIC VELOCITY FOR DISCRETE PARTICLES OF EARTH MATERIALS (v_b)





BASIC VELOCITIES FOR COHERENT EARTH MATERIALS (v_b)

ALLOWABLE VELOCITIES FOR UNPROTECTED EARTH CHANNELS

CHANNEL BOUNDARY MATERIALS	ALLOWABLE VELOCITY
DISCRETE PARTICLES	
Sediment Laden Flow	
$D_{75} > 0.4$ mm	Basic velocity chart value x D x A x B
$D_{75} < 0.4$ mm	2.0 fps
Sediment Free Flow	
$D_{75} > 2.0$ mm	Basic velocity chart value x D x A x B
$D_{75} < 2.0$ mm	2.0 fps
COHERENT EARTH MATERIALS	
PI > 10	Basic velocity chart value x D x A x F x C_e
PI < 10	2.0 fps

NOTES:

1. In no case should the allowable velocity be exceeded when the 10% chance discharge occurs, regardless of the design flow frequency.

FIGURE 6-2

ALLOWABLE VELOCITIES FOR UNPROTECTED EARTH CHANNELS

Revised -

has no effect on channel stability. Flows with concentrations lower than 1,000 ppm by weight are treated as sediment free flows.

2. Sediment laden flow is the condition in which the flow carries fine material in suspension at moderate to high concentrations so that stability is enhanced either through replacement of dislodged particles or through formation of a protective cover as the result of settling. Flows in this class carry sediment in suspension at concentrations equal or larger than 20,000 ppm by weight.

Estimation of the concentration of sediment in suspension is best made by sampling. See NEH, Section 3 for methods of sampling. If the concentration is not known from measurement, it can be estimated by the methods in Geologic Note 2.

Sediment transport rates are usually expressed in tons per day. To convert them into concentration use the equation:

$$C = 370 \frac{Q_s}{Q} \qquad \text{Eq. 6-1}$$

See page 4-41 of NEH 3, Chapter 4, for conversion from concentration in parts per million to milligrams per liter.

Depending on the type of soil, the effect of concentration of fine sediment (material smaller than 0.074 mm) in suspension on the allowable velocity is obtained from the curves on Figure 6-2.

If the suspended sediment concentration equals or exceeds 20,000 ppm by weight, use the sediment laden curve on Figure 6-2. If the suspended sediment concentration is 1,000 ppm or less by weight, use the sediment free curve on Figure 6-2. A linear interpolation may be made between these curves for suspended sediment concentrations between 1,000 ppm and 20,000 ppm.

Adjustment in the basic velocity to reflect the modifying effects of frequency of runoff, curvature in alignment, bank slopes, density of bed and bank materials, and depth of flow are made using the adjustment curves on Figure 6-2.

The alignment factor, A, and the depth factor, D, apply to all soil conditions. The bank slope factor, B, applies only to channels in soils that behave as discrete particles. The frequency correction, F, applies only to channels in soils that resist erosion as a coherent mass. The density correction factor, C_e, applies to all soil materials except clean sands and gravels (containing less than 5 percent material passing size #200).

Figure 6-2 gives the correction factors (F) for frequencies of occurrence lower than 10 percent. Channels designed for less frequent flows using this correction factor should be designed to be stable at the 10 percent chance frequency discharge as well as at the design discharge.

If the soils along the channel boundary behave as discrete particles with D_{75} larger than 0.4 mm for sediment laden flow or larger than 2.0 mm for sediment free flow, the allowable velocity is determined by adjusting the basic velocity read from the curves on Figure 6-2 for the effects of alignment, bank slope, and depth. If the soils behave as discrete particles and D_{75} is smaller than 0.4 mm for sediment laden flow or 2.0 mm for sediment free flow the allowable velocity is 2.0 fps. For channels in these soils no adjustments are to be made to the basic velocity of 2.0 fps.

In cases where the soils in the channel boundary resist erosion as a coherent mass, the allowable velocity is determined by adjusting the basic velocity from Figure 6-2 for the effects of depth, alignment, bank slope, frequency of occurrence of design flow, and for the density of the boundary soil materials.

Design Procedure for Allowable Velocity Approach

The use of the allowable velocity approach in checking the stability of earth channels involves the following steps:

1. Determine the hydraulics of the system. This includes hydrologic determinations as well as the stage-discharge relationships for the channel considered. The procedures to be used in this step are included in Chapter 4 and Chapter 5 of this Technical Release.

2. Determine the properties of the earth materials forming the banks and bed of the design reach and of the channel upstream.

3. Determine sediment yield to reach and calculate sediment concentration for design flow.

4. Check to see if the allowable velocity procedure is applicable. Use Figure 6-1.

5. Compare the design velocities with the allowable velocities from Figure 6-2 for the materials forming the channel boundary.

6. If the allowable velocities are less than design velocities, it may be necessary to consider a mobile boundary condition and evaluate the channel using appropriate sediment transport theory.

Examples of Allowable Velocity Approach

Example 6-1

Given: A channel is to be constructed to convey the flow from a 2 percent chance flood through an intensively cultivated area. The hydraulics of the system indicate that a trapezoidal channel with 2:1 side slopes and a 40 foot bottom width will carry the design flow at a depth of 8.7 feet and a velocity of 5.45 fps. Soil investigations reveal that the channel will be excavated in a moderately rounded clean sandy gravel with a D_{75} size of 2.25 inches. Sampling of soils in the drainage area and estimate of erosion and sediment yield indicate that on an average annual basis approximately 1000 tons of sediment finer than 1.0 mm. and 20 tons of material coarser than 1.0 mm are available for transport in channel. The amount of abrasion resulting from the transporting of this small amount of sediment coarser than 1.0 mm. is considered insignificant. Sediment transport computations indicate all of the sediment supplied to the channel will be transported through the reach. The sediment transport and hydrologic evaluations indicate the design flow will transport the available sediment at a concentration of about 500 ppm. The channel is straight except for one curve with a radius of 600 feet.

Determine:

1. The allowable velocity, V_a
2. The stability of the reach.

Solution: Determine basic velocity from Figure 6-2, sediment free curve because sediment concentration of 500 ppm is less than 1,000 ppm.

V_b = 6.7 fps

Depth correction factor, D = 1.22 (from Figure 6-2)

Bank slope correction, B = 0.72 (from Figure 6-2)

Alignment correction A

$$\frac{\text{curve radius}}{\text{water surface width}} = \frac{600}{74.8} = 8.02$$

A = 0.89 (from Figure 6-2)

Density correction, C_e, does not apply

Frequency correction, F, does not apply

$V_a = V_bDB = (6.7)(1.22)(0.72)$ straight reaches

$= 5.88$ fps

$V_a = V_bDBA = (6.7)(1.22)(0.72)(0.89)$ curved reach

$= 5.24$ fps

The proposed design velocity of 5.45 fps is less than V_a = 5.88 fps in the straight reaches but greater than V_a = 5.24 fps in the curved reaches. Either the channel alignment or geometry needs to be altered or the curve needs structural protection.

Example 6-2

Given: A channel is to be constructed to convey the flow from a 2 percent chance flood through an intensively cultivated area. The hydraulics of the system indicate that a trapezoidal channel with 2:1 side slopes and a 40 foot bottom width will carry the design flow at a depth of 8.7 feet and a velocity of 5.45 fps. The channel is to be excavated into a silty clay CL soil with a Plasticity Index of 18, a dry density of 92 pcf, and a specific gravity of 2.71. Sediment transport evaluations indicate the design flow will have a fairly stable sediment concentration of about 500 ppm. with essentially no bed material load larger than 1.0 mm. The channel is straight except for one curve with a radius of 600 feet. The 10 percent chance flood results in a depth of flow of 7.4 feet and a velocity of 4.93 fps.

Determine:

1. The allowable velocity, V_a

2. The stability of the reach.

Solution: Sediment concentration of 500 ppm is less than 1,000 ppm therefore it is classed as sediment free flow.

V_b = 3.7 fps (from Figure 6-2)

for the 2 percent chance flood

Depth correction, D = 1.22 (from Figure 6-2)

Density correction, compute e.

$$e = G \frac{\gamma_w}{\gamma_d} - 1 = \frac{(2.71)(62.4)}{92} - 1 = 0.83$$

C_e = 1.0 (from Figure 6-2)

Frequency correction, F = 1.5 (from Figure 6-2)

Alignment correction A

$$\frac{\text{Curve radius}}{\text{water surface width}} = \frac{600}{74.8} = 8.02$$

A = 0.89 (from Figure 6-2)

$V_a = V_b D C_e F$ Straight reach

$V_a = (3.7)(1.22)(1.0)(1.5) = 6.77$ fps

$V_a = V_b D C_e F A$ Curved reach

$V_a = (3.7)(1.22)(1.0)(1.5)(0.89) = 6.03$ fps

The design velocity is less than the allowable velocity for the 2 percent chance flow. Check the 10 percent chance flow velocity with no frequency correction against the allowable velocity for the 10 percent chance flow.

$V_a = V_b D C_e$ Straight reaches

$V_a = (3.7)(1.19)(1.0) = 4.40$ fps

$V_a = V_b D C_e A$ Curved reaches

$V_a = (3.7)(1.19)(1.0)(0.90) = 3.96$ fps

The allowable velocity with no frequency correction is exceeded by the 10 percent chance flow velocity. An evaluation should be made to estimate the magnitude of scour or possible depth of scour before an armor is formed (See page 6-30). Using this procedure in conjunction with the appropriate sediment transport equations, the magnitude of instability can be evaluated. Channel alignment, slope, or geometry must be altered or the channel must be protected.

Example 6-3

Given: The same conditions as in Example 6-1 except that the suspended sediment concentration is 30,000 ppm.

Determine:

1. The allowable velocity, V_a

2. The stability of the reach.

Solution: The suspended sediment concentration of 30,000 ppm is greater than 20,000 ppm.

Therefore it is classed as sediment laden flow.

$V_b = 9.0$ fps (from Figure 6-2)

Depth correction factor D = 1.22 (From Figure 6-2)

Bank Slope correction factor B = 0.72 (From Figure 6-2)

Alignment correction factor A = 0.89 (From Figure 6-2)

Density correction factor, C_e does not apply

Frequency correction factor F, does not apply

$$V_a = V_bDB = (9.0)(1.22)(0.72) = 7.91 \text{ fps for straight reaches}$$

$$V_a = V_bDBA = (9.0)(1.22)(0.72)(0.89) = 7.04 \text{ fps for the curved reach}$$

The proposed design velocity of 5.45 fps is less than V_a = 7.91 fps, straight reaches and V_a = 7.04 fps in the curved reaches. This reach of channel is considered to be stable relative to scour. Use sediment transport equations to determine the possibility of channel aggradation.

Example 6-4

Given:

1. Trapezoidal channel to convey the 50 percent chance flood at bank full flow.

2. The 10-year peak discharge exceeds the 2-year peak by 30 percent; the 10-year flow for as-build conditions exceeds bank full capacity.

3. From design hydraulic calculations:

   Bottom Width = 30 ft.
   Flow Depth = 9.0 ft. (bank full)
   Side Slopes = 2:1
   n (aged condition) = 0.030
   n (as-built condition) = 0.025
   Velocity (aged condition) = 3.3 fps (bank full)

4. Sharpest Curve = 350 ft. radius.

5. Estimated sediment concentration at design discharge = 2000 ppm.

6. Two layers of soil material are to be evaluated for stability. The upper layer is classified CL; the plasticity index is 15 and the void ratio is 0.9. The lower layer is classified as a GM with a D_{75} particle size of 10 mm.

Determine:

1. The allowable velocity for the CL and GM materials.

2. The stability of the channel.

Solution:

1. CL Layer

   From Figure 6-2 for coherent particles, the allowable velocity = (basic velocity)$DAFC_e$

Basic velocity for sediment laden flow (20,000 ppm) = 4.75 fps.

Basic velocity for sediment free flow (1000 ppm) = 3.25 fps.

Basic velocity for 2000 ppm sediment concentration (by linear interpolation) = 3.33 fps.

Computations for correction factor A for sharpest curve:

$$\frac{\text{Curve radius}}{\text{Water surface width}} = \frac{350}{66} = 5.3$$

Correction factor F = 1.0
Correction factor A = 0.75
Correction factor D = 1.23
Correction factor C_e = 0.97

Allowable velocity for straight channel = $V_a = V_b DAFC_e$
$V_a = (3.33)(1.23)(1.0)(1.0)(0.97) = 3.97$ fps

This is greater than the 3.3 fps design value and the channel is stable for this condition.

Allowable velocity for sharpest curve = $V_a = V_b DAFC_e$
$V_a = (3.33)(1.23)(0.75)(1.0)(0.97) = 2.98$ fps

This is less than the 3.3 fps design value and the channel is not stable for the sharpest curve.

2. GM Layer

From Figure 6-2, the allowable velocity is $V_a = (V_b)$ DAB

V_b for sediment laden flow (20,000 ppm) = 5.3 ft/sec

V_b for sediment free flow (1,000 ppm) = 3.4 ft/sec

V_b for 2000 ppm sediment concentration (by linear interpolation) = 3.5 ft/sec

Correction factor A = 0.75 (R = 350')
Correction factor D = 1.23
Correction factor B = 0.71

Allowable velocity for straight channel
$V_a = V_b DAB = (3.5)(1.23)(1.0)(0.71) = 3.06$ fps

Allowable velocity for sharpest curve
$V_a = V_b$ DAB = (3.5)(1.25)(0.75)(0.71) = 2.29 fps

Summary:

The upper layer (CL) is stable for the straight sections and unstable for curve with a radius of 350 ft.

The lower layer (GM) is unstable for both the straight and curved sections.

This condition may need additional evaluation using the appropriate sediment transport equations.

Tractive Stress Approach

General

The tractive force is the tangential pull of flowing water on the wetted channel boundary; it is equal to the total friction force that resists flow but acts in the opposite direction. Tractive stress is the tractive force per unit area of the boundary. The tractive force is expressed in units of pounds, while tractive stress is expressed in units of pounds per square foot. The tractive force in a prismatic channel reach is equal to the weight of the fluid prism multiplied by the energy gradient.

The tractive stress approach to channel stability analysis provides a method to evaluate the stress at the interface between flowing water and the materials in the channel boundary.

The method for obtaining the design or actual tractive stress acting on the bed or sides of a channel and the allowable tractive stress depends on the D_{75} size of the materials involved. When coarse grained discrete particle soils are involved Lane's[25] method is used. When fine grained soils are involved, a method derived from the work of Keulegan and modified by Einstein[27], and Vanoni and Brooks[48] is used. The separation size for this determination is D_{75} = 1/4 inch.

Coarse-grained Discrete Particle Soils - D_{75} > 1/4 inch - Lane's Method

A. Determination of Actual Tractive Stress

1. Actual tractive stress in an infinitely wide channel.

Generally, Manning's roughness coefficient n reflects the overall impedence to flow including grain roughness, form roughness, vegetation, curved alignment, etc. Lane's[25] work showed that for soils with a D_{75} size between 0.25" (6.35mm) and 5.0" (127mm) the value of Manning's coefficient n resulting from the roughness of the soil particles is determined by:

$$n_t = \frac{D_{75}^{1/6}}{39} \quad \text{with } D_{75} \text{ expressed in inches (Eq. 6-2)}$$

The value of n_t determined by equation 6-2 represents the retardance to flow caused by roughness of the soil grains.

Use n_t from equation 6-2 as a first step in the procedure in NEH, Section 5, Supplement B to determine the total value of Manning's coefficient. The value of n_t from equation 6-2 can be used next in equation 6-3 to compute s_t, the friction gradient associated with the particular boundary material being considered.

$$s_t = \left(\frac{n_t}{n}\right)^2 s_e \qquad \text{(Eq. 6-3)}$$

The tractive stress acting on the soil grains in an infinitely wide channel is found by:

$$\tau_\infty = \gamma_w \, d s_t \qquad \text{(Eq. 6-4)}$$

where the terms are as defined in the glossary.

2. Distribution of the tractive stress along the channel perimeter:

   In open channels the tractive stresses are not distributed uniformly along the perimeter. Laboratory experiments and field observations have indicated that in trapezoidal channels the stresses are very small near the water surface and near the corners of the channel and assume their maximum value near the center of the bed. The maximum value on the banks occurs near the lower third point.

   Figure 6-3 and 6-4 give the maximum tractive stresses in a trapezoidal channel in relation to the tractive stress in an infinitely wide channel having the same depth of flow and value of s_t.

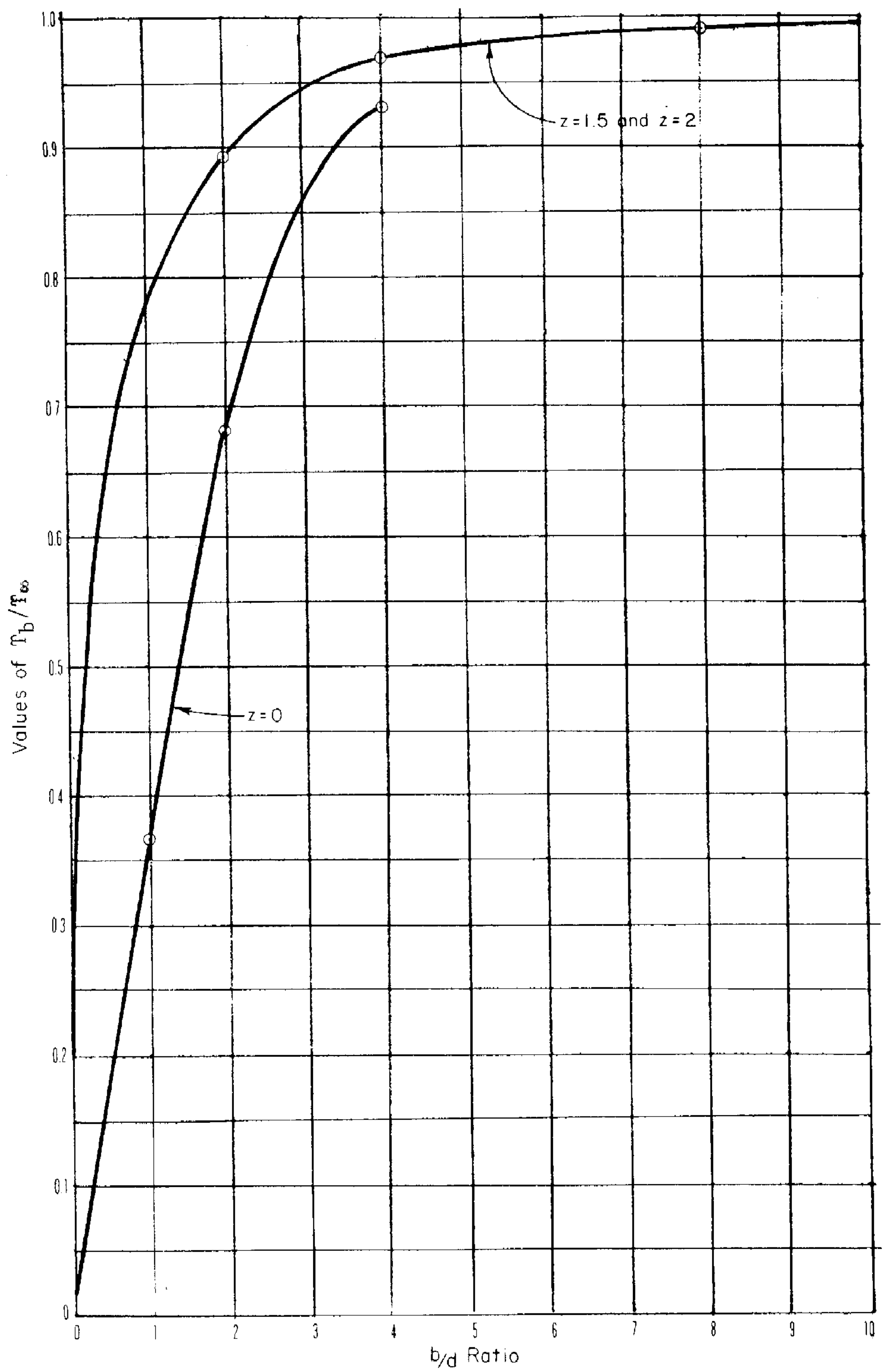
3. Tractive stresses on curved reaches:

   Curves in channels cause the maximum tractive stresses to increase above those in straight channels. The maximum tractive stresses in a channel with a single curve occur on the inside bank in the upstream portion of the curve and near the outer bank downstream from the curve. Compounding of curves in a channel complicates the flow pattern and causes a compounding of the maximum tractive stresses.

   Figure 6-5 gives values of maximum tractive stresses based on judgment coupled with very limited experimental data. It does not show the effect of depth of flow and length of curve and its use is only justified until more accurate information is obtained. Figure 6-6 with a similar degree of accuracy, gives

247-927 O - 78 - 11

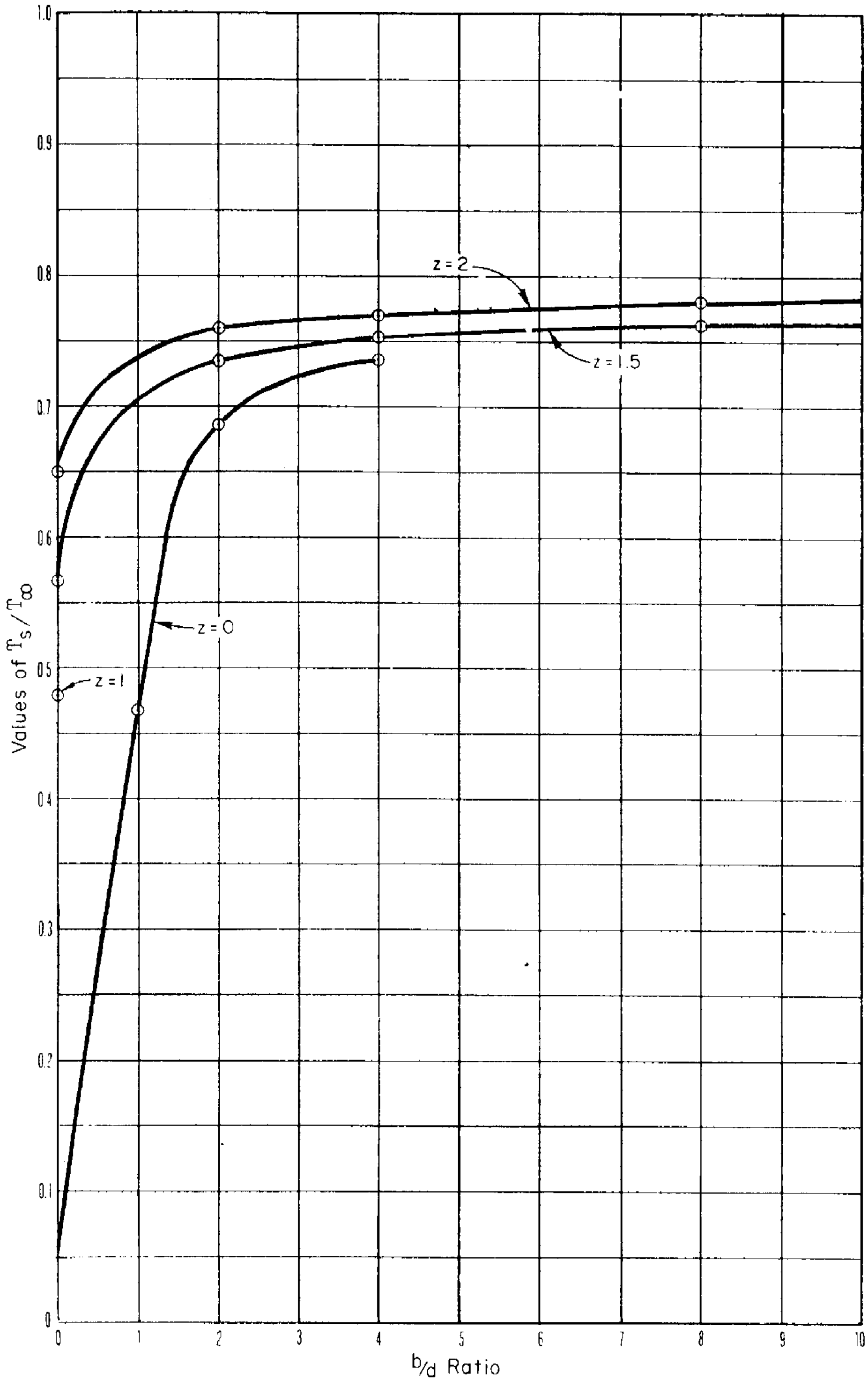
HYDRAULICS: Channel stability; actual maximum tractive stress, τ_b, on bed of straight trapezoidal channels



REFERENCE
Bureau Of Reclamation "Progress Report On Results Of Studies On Design Of Stable Channels" Hyd-352

FIGURE 6-3

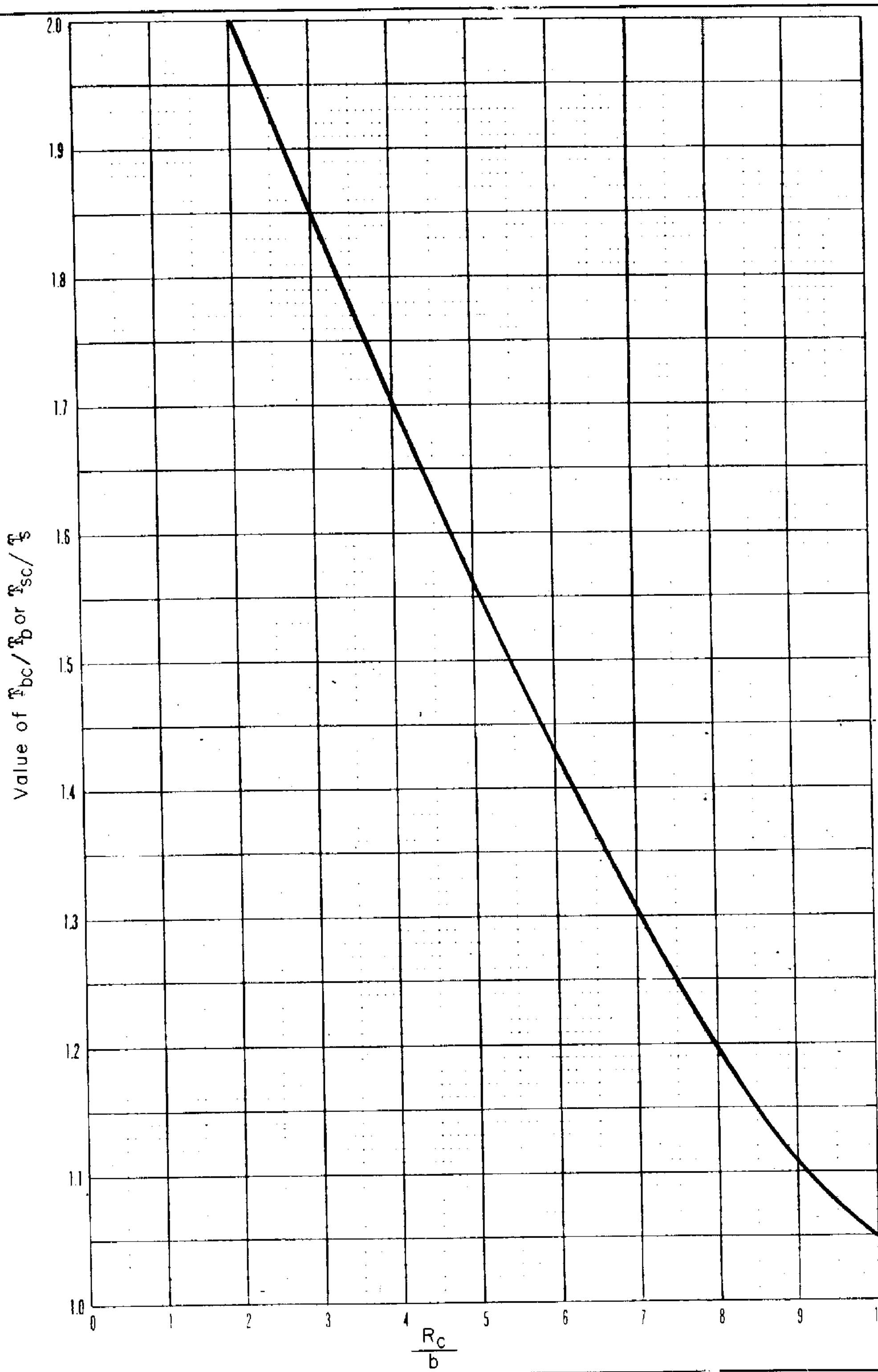
HYDRAULICS: Channel stability; actual maximum tractive stress, T_s, on sides of straight trapezoidal channels



REFERENCE
Bureau Of Reclamation "Progress Report Of Results Of Studies On Design Of Stable Channels" Hyd-352

FIGURE 6-4

HYDRAULICS: Channel stability; actual maximum tractive stress, τ_{bc} and τ_{sc}, on bed and sides of trapezoidal channels within a curved reach



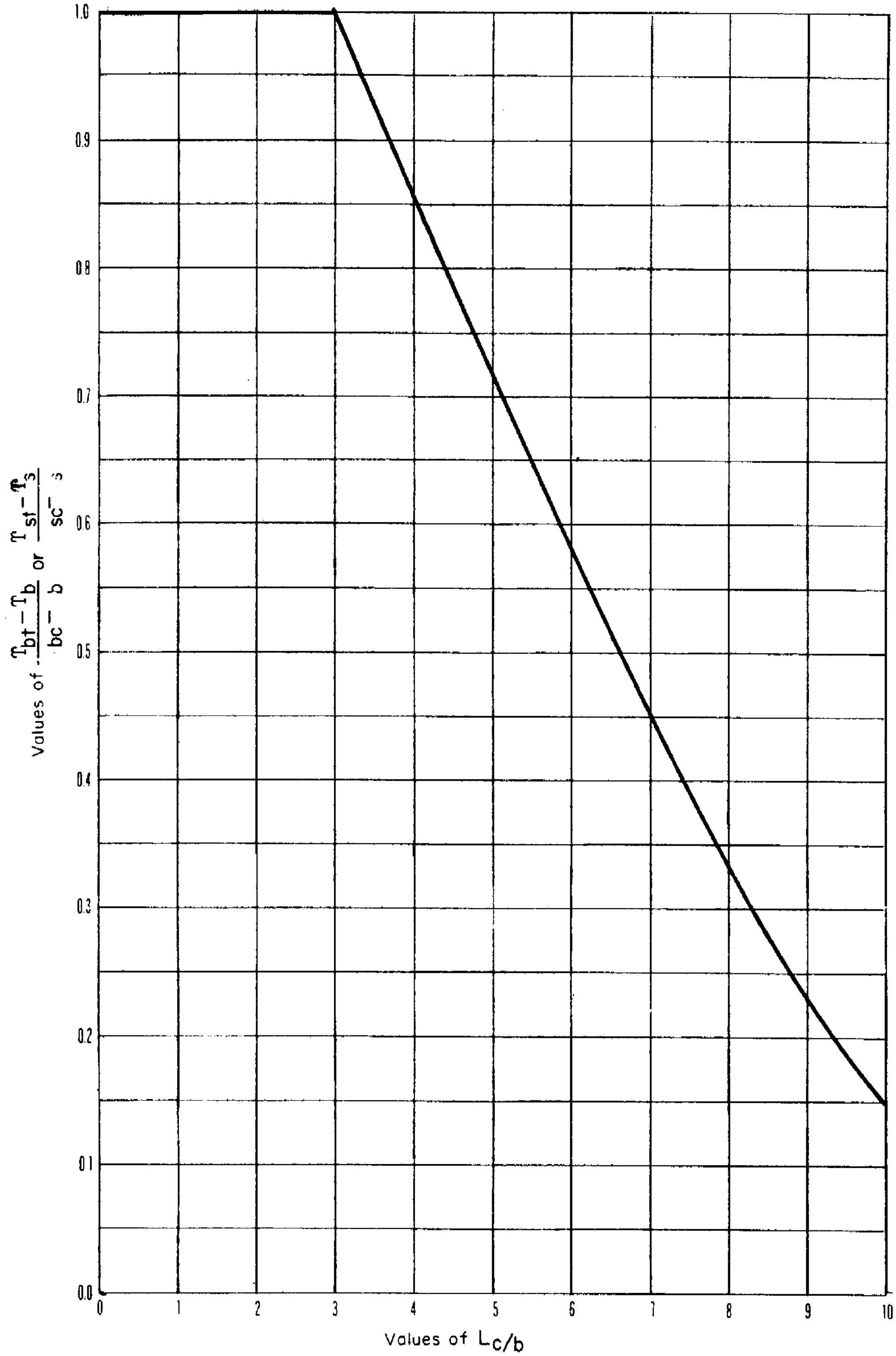
REFERENCE

Lane, Emory W., Design of Stable Channels Transaction, A S C E , vol 120, 1955

Nece, R E., Givler, C A , and Drinker, P A , Measurement of Boundary Shear Stress in an Open Channel Curve with a Suface Pitot Tube M I T Tech note (no 6), August 1959

FIGURE 6-5

HYDRAULICS: Channel stability; actual maximum tractive stresses γ_{bt} and γ_{st}, on bed and sides of trapezoidal channels in straight reaches immediately downstream from curved reaches



REFERENCE
Nece, R.E., Givler, G.A., and Drinker, P.A., Measurement of Boundary Shear Stress in an Open Curve Channel with a Surface Pitot Tube: M.I.T. Tech. note (no. 6), Aug. 1959

FIGURE 6-6

the maximum tractive stresses at various distances downstream from the curve.

B. Allowable Tractive Stress

The allowable tractive stress for channel beds, τ_{Lb}, composed of soil particles with discrete, single grain behavior with a given D_{75} is:

$$\tau_{Lb} = 0.4\ D_{75}$$

When $0.25 \text{ in.} < D_{75} < 5.0 \text{ in.}$ (Eq. 6-5)

The allowable tractive stress for channel sides τ_{Ls} is less than that of the same material in the bed of the channel because the gravity force aids the tractive stress in moving the materials. The allowable tractive stress for channel sides composed of soil particles behaving as discrete single grain materials, considering the effect of the side slope z and the angle of repose ϕ_R with the horizontal is

$$\tau_{Ls} = 0.4\ K\ D_{75} \quad . \ . \ . \ . \quad 0.25 \text{ in.} < D_{75} < 5.0 \text{ in.} \qquad \text{(Eq. 6-6)}$$

$$K = \sqrt{\frac{z^2 - \cot^2 \phi_R}{1 + z^2}} \quad . \ . \ . \ . \ . \qquad \text{(Eq. 6-7)}$$

Figure 6-7 gives an evaluation of the angles of repose corresponding to the degree of angularity of the material. Figure 6-8 gives values of K from equation 6-7.

When the unit weight γ_s of the constituents of the material having a grain size larger than the D_{75} size is significantly different than 160 lb/ft^3, the limiting tractive stress τ_{Lb} and τ_{Ls} as given by equations (6-5) and (6-6) should be multiplied by the factor.

$$T = \frac{\gamma_s - \gamma_w}{97.6} \qquad \text{(Eq. 6-8)}$$

<u>Fine Grained Soils - D_{75} < 1/4 inch</u>

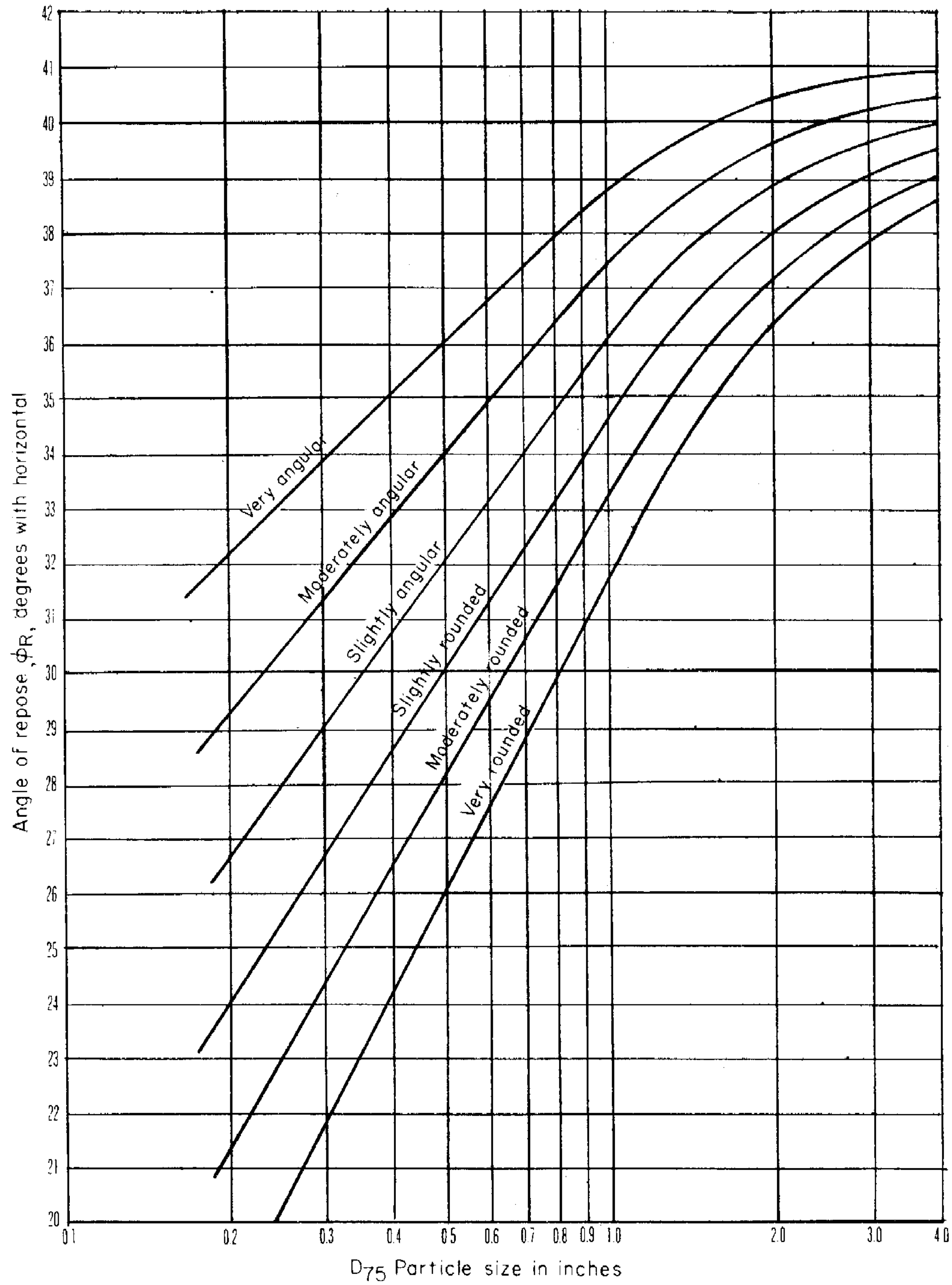
A. Determination of Actual Tractive Stress

1. Reference tractive stress

The expression for the reference tractive stress is:

$$\tau = \gamma_w R_t s_e \qquad \text{(Eq. 6-9)}$$

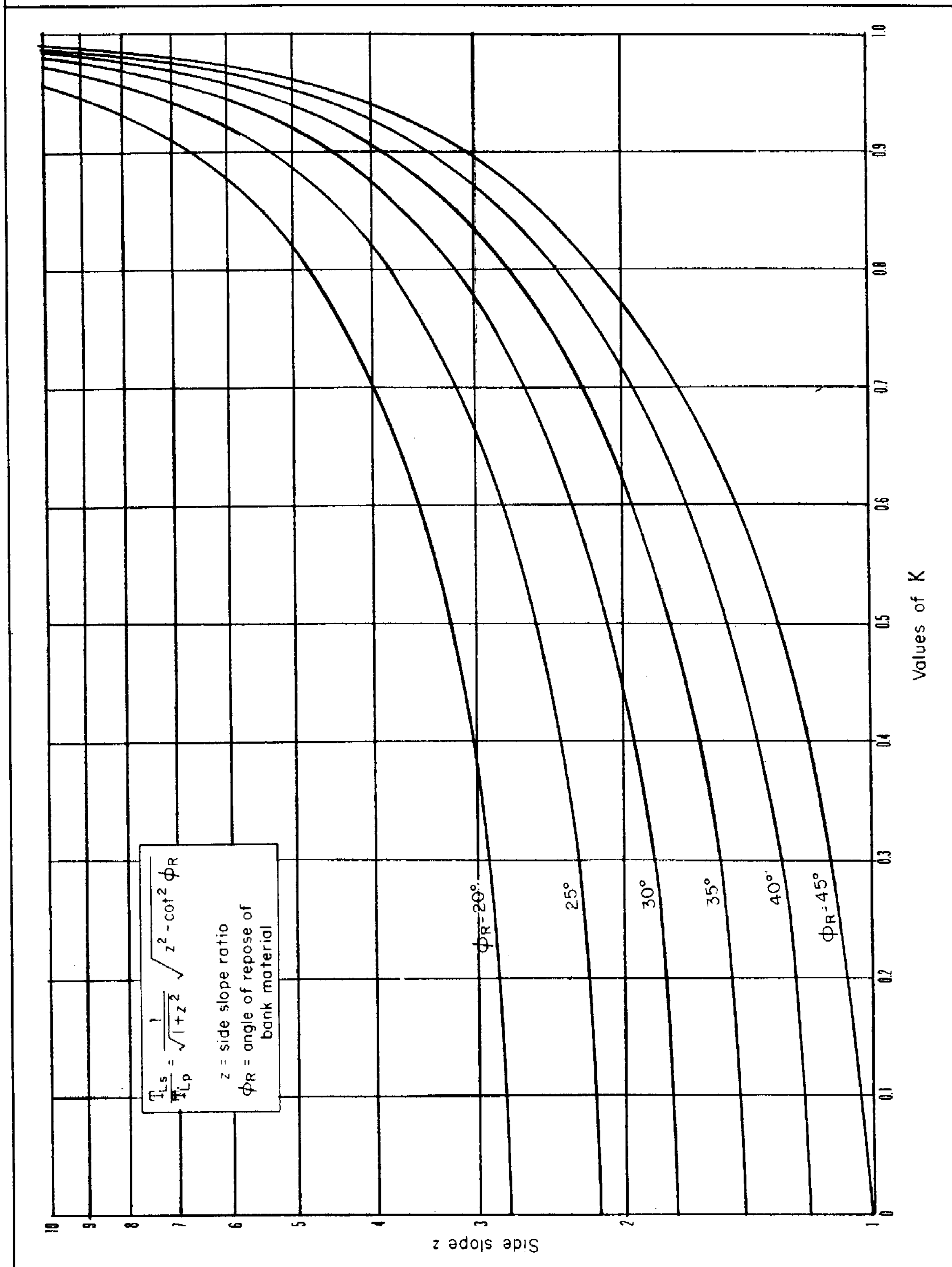
HYDRAULICS: Channel stability; angle of respose, ϕ_R, for non-cohesive materials



REFERENCE
Bureau Of Reclamation "Progress Report Of Results Of Studies On Design Of Stable Channels" Hyd-352

FIGURE 6-7

HYDRAULICS: Channel stability; limiting tractive stress τ_{Ls} for sides of trapezoidal channels having non-cohesive materials



REFERENCE
Bureau Of Reclamation "Progress Report On Results Of Studies On Design Of Studies Stable Channels" Hyd-352

FIGURE 6-8

In a given situation γ and s_e are known so that the only unknown is R_t. The value of R_t can be determined from the logarithmic frictional formula developed by Keulegan and modified by Einstein.[27]

$$\frac{V}{\sqrt{g\ R_t\ s_e}} = 5.75 \log \left(12.27\ \frac{R_t\ x}{k_s}\right) \qquad \text{(Eq. 6-10)}$$

k_s is the D_{65} size in ft.

The factor x in equation 6-10 describes the effect on the frictional resistance of the ratio of the characteristic roughness length k_s to the thickness of the laminar sublayer δ. This thickness is determined from the equation

$$\delta = \frac{11.6\ \nu}{\sqrt{g\ R_t\ s_e}} \qquad \text{(Eq. 6-11)}$$

A relationship between x and k_s/δ has been developed empirically by Einstein[27] and represented by a curve. With the help of this curve and equations 6-10 and 6-11 the value of R_t can be determined provided that V, s_e, k_s and the temperature of the water are known. The computational solution for R_t follows an iterative procedure which is rather involved. A simpler graphical solution has been developed by Vanoni and Brooks[48] and the basic family of curves that constitute it, is shown in Figure 6-9. Figure 6-10 shows the extension of the curves outside the region covered in the original publication.

Figure 6-11 gives curves from which values of density ρ and kinematic viscosity of the water ν can be obtained.

The computation of reference tractive stress (τ) is facilitated by following the procedure on page 6-28.

2. Distribution of the tractive stress along the channel perimeter:

   In open channels the tractive stresses are not distributed uniformly along the perimeter. Laboratory experiments and field observations have indicated that in trapezoidal channels the stresses are very small near the water surface and near the corners of the channel and assume their maximum value near the center of the bed. The maximum value on the banks occurs near the lower third point.

   The graphs in Figures 6-12 and 6-13 may be used to evaluate maximum stress values on the banks and the bed respectively. These figures are to be used along with τ, the reference tractive stress, to obtain values for the maximum tractive stress on the sides and bed of trapezoidal channels in fine grained soils.

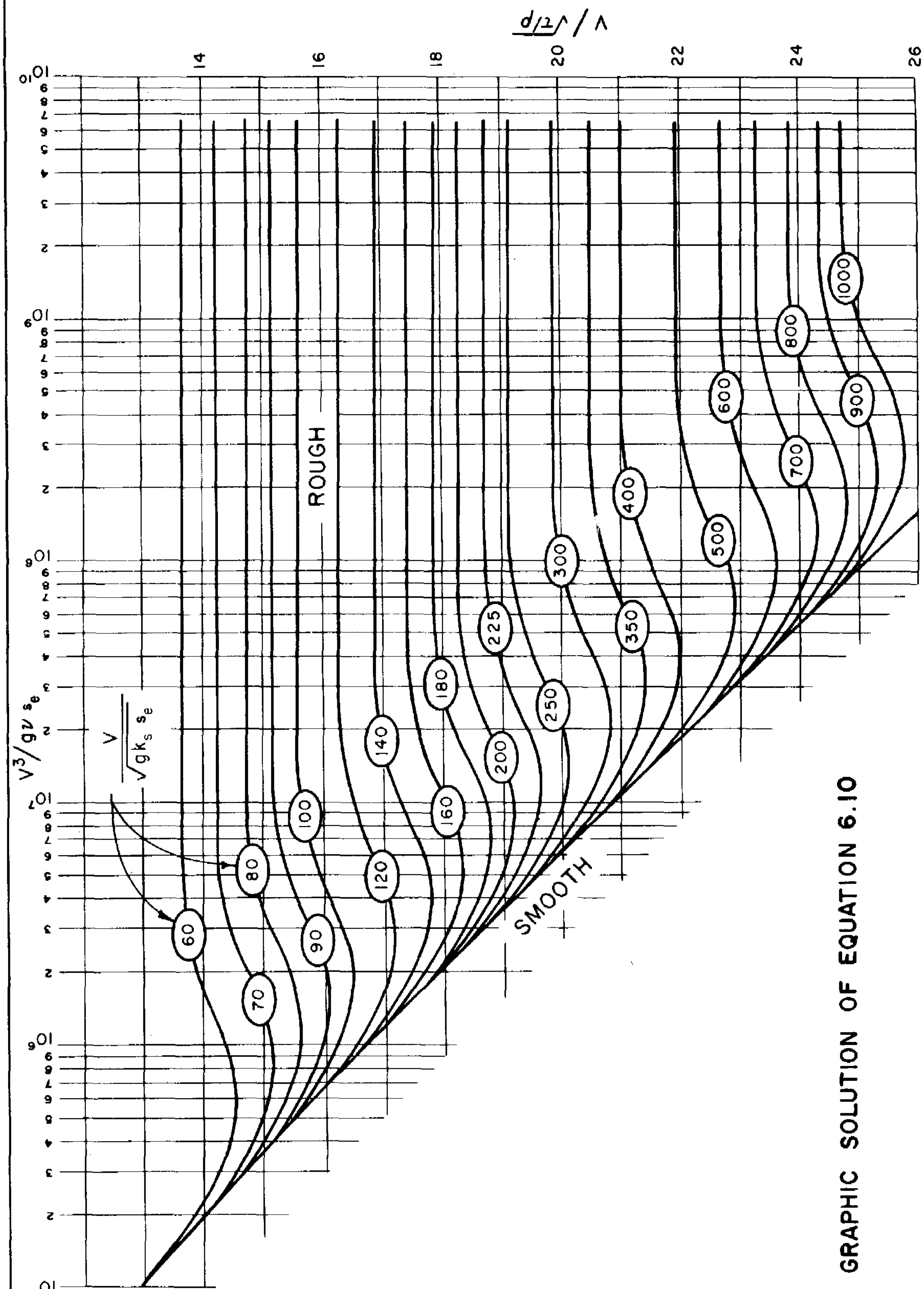
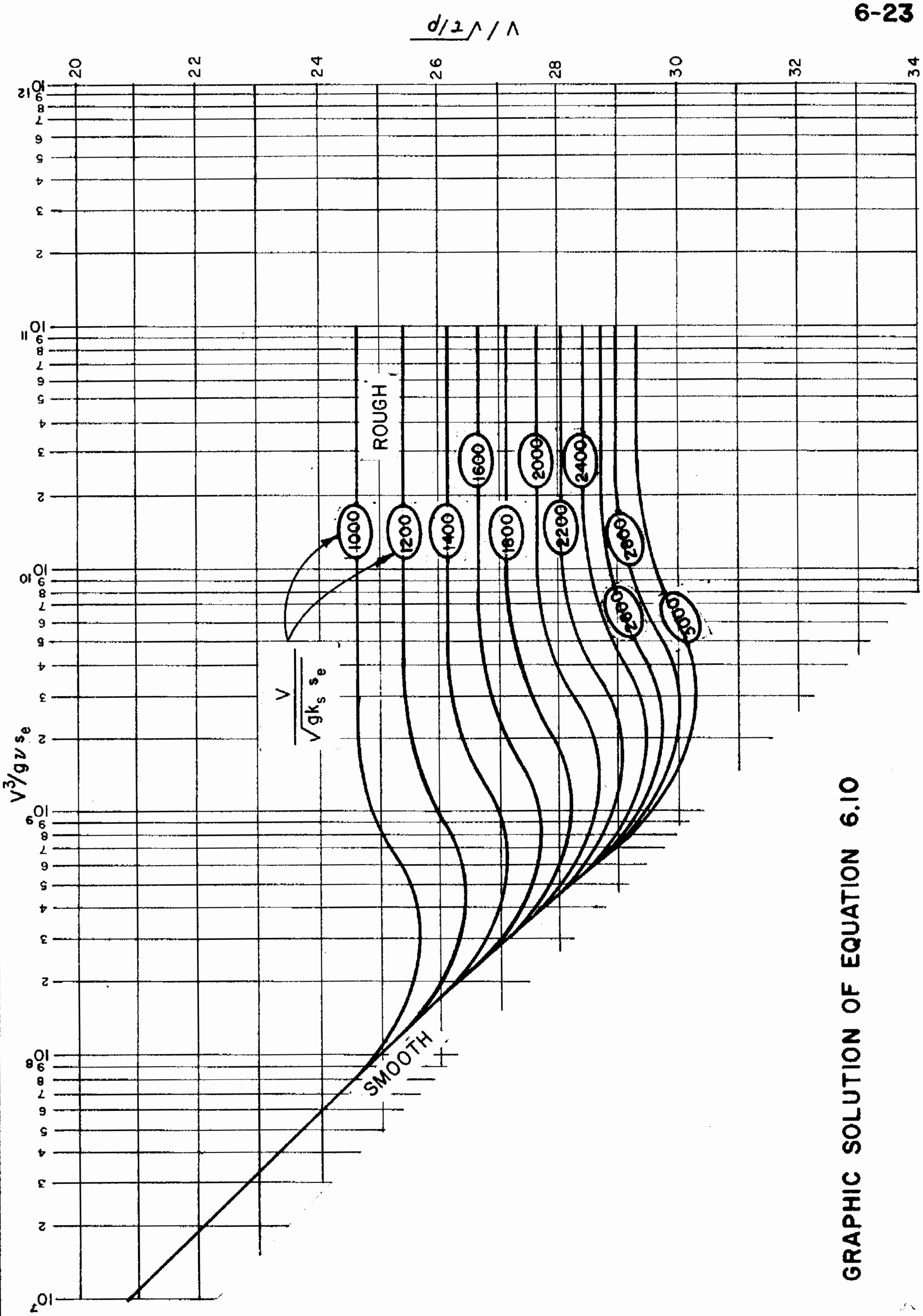


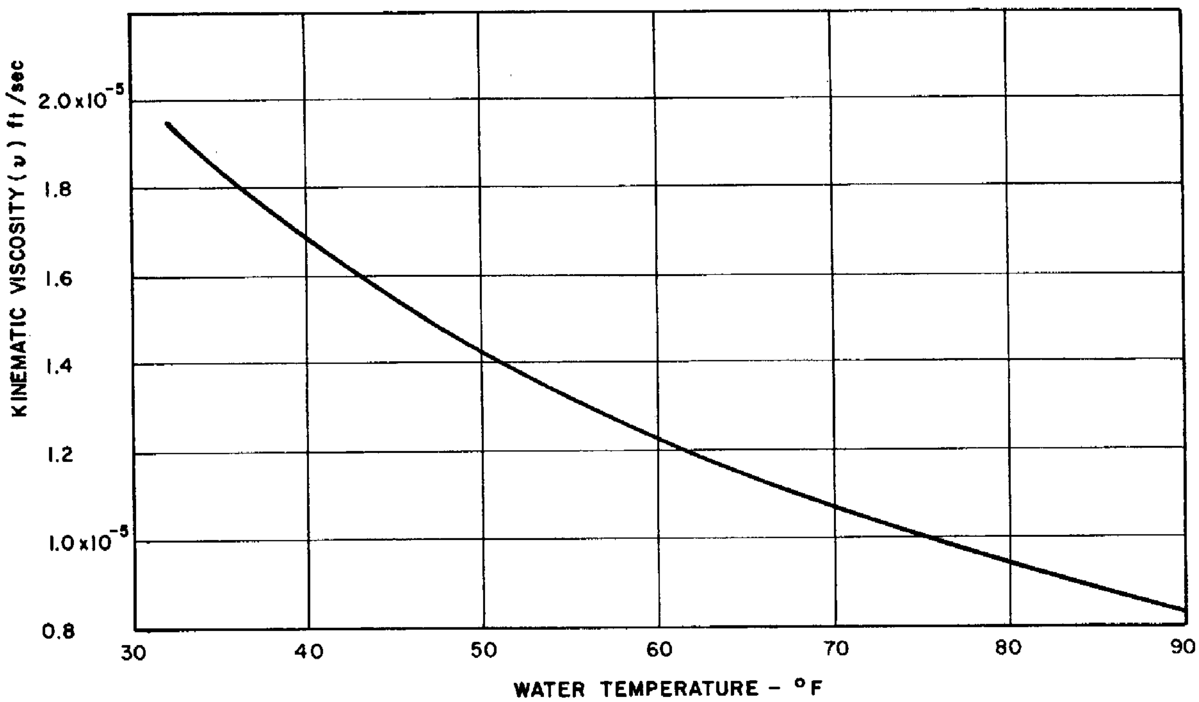
FIGURE 6-9

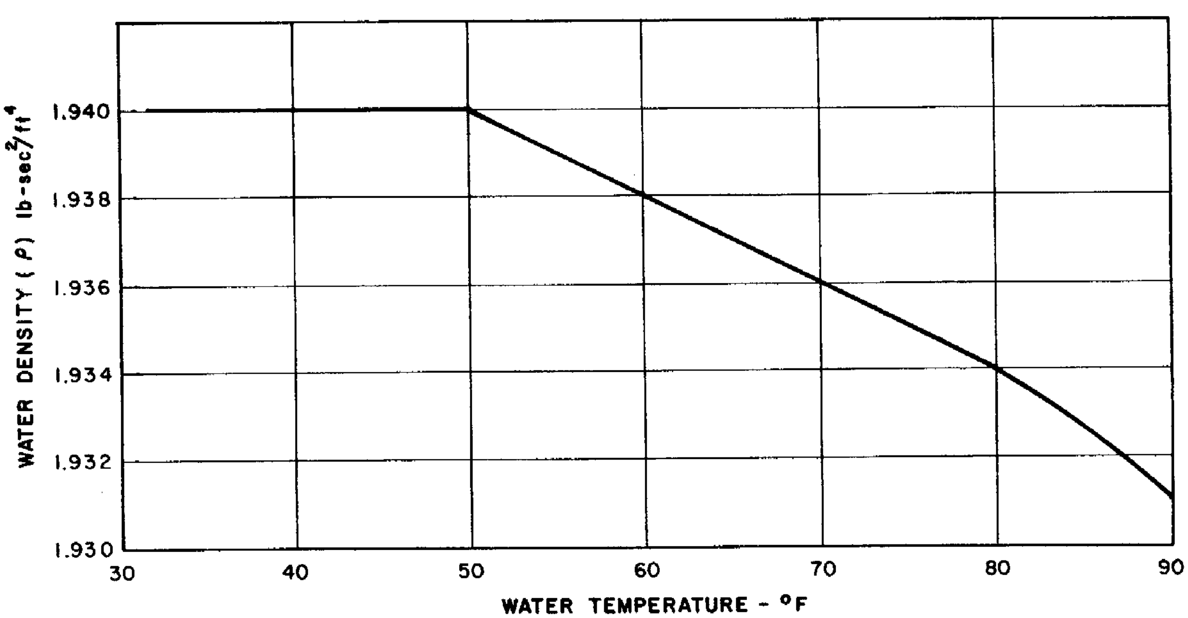
GRAPHIC SOLUTION OF EQUATION 6.10



GRAPHIC SOLUTION OF EQUATION 6.10

FIGURE 6-10





Values of ρ and ν for various water temperatures

FIGURE 6-11

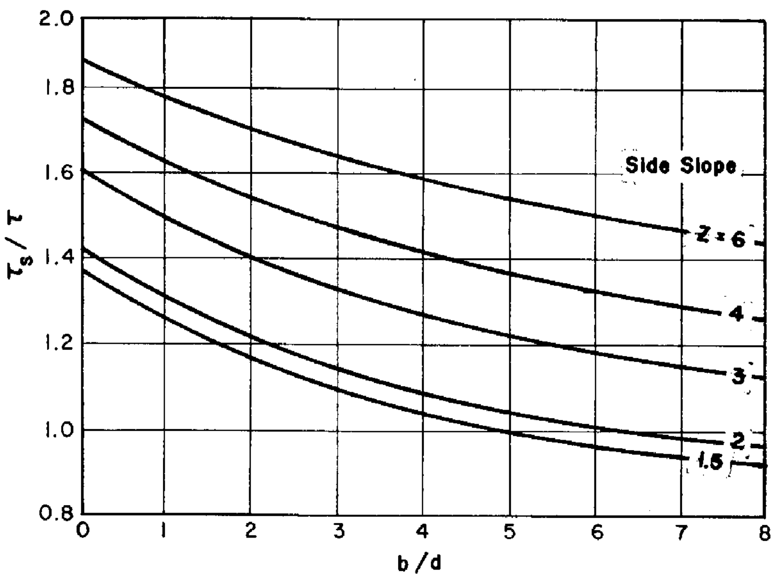


FIGURE 6-12 : Applied Maximum Tractive Stresses, τ_s, On Sides Of Straight Trapezoidal Channels.

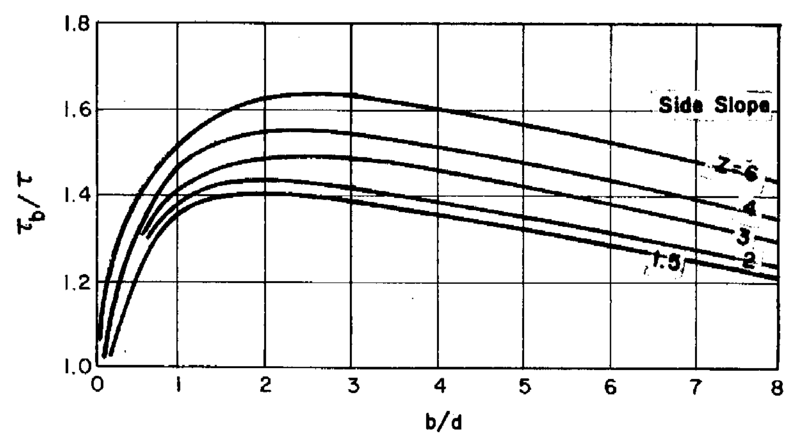


FIGURE 6-13 : Applied Maximum Tractive Stresses, τ_b, On Bed Of Straight Trapezoidal Channels.

Curves reproduced from " Tentative Design Procedure for Riprap - Lined Channels "National Cooperative Highway Research Program. Report No. 108.

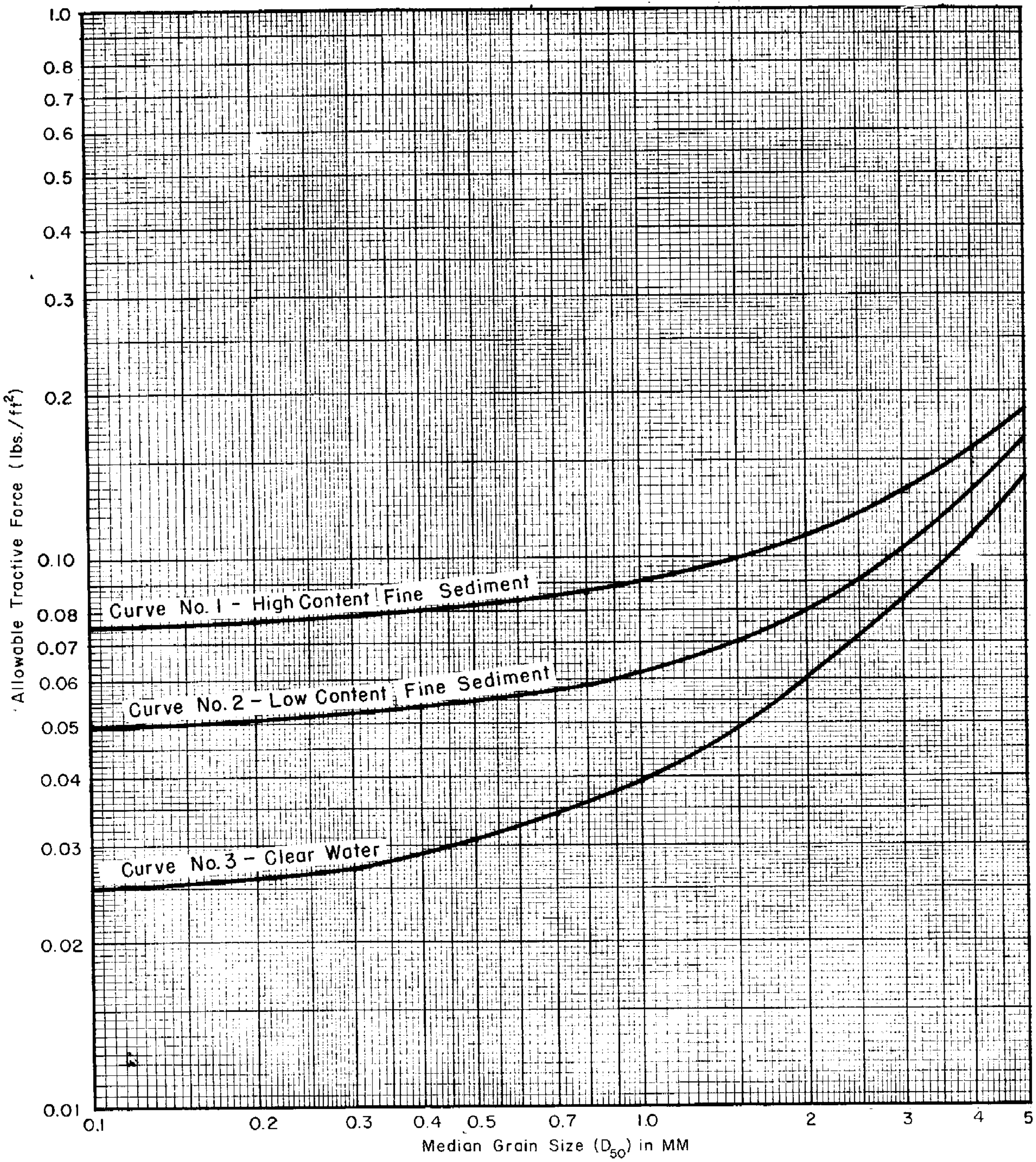


FIGURE 6-14

ALLOWABLE TRACTIVE STRESS

NON-COHESIVE SOILS, $D_{75} \leq 0.25"$

REFERENCE: LANE, E.W., "DESIGN OF STABLE CHANNELS", TRANSACTIONS A.S.C.E., VOLUME 120

3. Tractive stresses in curved reaches:

   Figures 6-5 and 6-6 used to determine the maximum tractive stresses in curved reaches for coarse grained soils may also be used to obtain these values for fine grained soils. The values for the maximum tractive stresses on the beds and sides as determined above are used in conjunction with these charts to obtain values for curved reaches.

B. Allowable Tractive Stresses - Fine grained soils

The stability of channels in fine grained soils (D_{75} <0.25") may be checked using the curves in Figure 6-14. These curves were developed by Lane[25/]. The curves relate the median grain size of the soils to the allowable tractive stress. Curve 1 is to be used when the stream under consideration carries a load of 20,000 ppm by weight or more of fine suspended sediment. Curve 2 is to be used for streams carrying up to 2,000 ppm by weight of fine suspended sediment. Curve 3 is for sediment free flows (less than 1,000 ppm).

When the value of D_{50} for fine grained soils is greater than 5 mm use the allowable tractive stress values shown on the chart for 5 mm.

For values of D_{50} less than those shown on the chart (0.1mm) use the allowable tractive stress values for 0.1 mm. However, if this is done 0.1 mm should be used as the D_{65} size in obtaining the reference tractive stress.

<u>Procedures - Tractive Stress Approach</u>

The use of tractive stress to check the ability of earth channels to resist erosive stresses involves the following steps:

1. Determine the hydraulics of the channel. This includes hydrologic determinations as well as the stage-discharge relationships for the channel being considered. The procedures to be used in making these determinations are included in Chapter 4 and Chapter 5 of this Technical Release.

2. Determine sediment yield to reach and calculate sediment concentration for design flow.

3. Determine the character of the earth materials in the boundary of the channel.

4. Check to see if the tractive stress approach is applicable. Use Figure 6-1.

5. Compute the tractive stresses exerted by the flowing water on the boundary of the channel being studied. Use the proper procedure as established by the D_{75} size of the materials.

6. Check the ability of the soil materials forming the channel to resist the computed tractive stresses.

The computation for the reference tractive stress for fine grained soils is facilitated by using the following procedure:

1. Determine s_e and V: Evaluate Manning's n by the method described in NEH-5, Supplement B.

2. Enter the graphs in Figure 6-11 with the value of temperature in °F and read the density ρ and the kinematic viscosity of the water ν.

3. Compute $\dfrac{V^3}{g\nu s_e}$.

4. Compute $\dfrac{V}{\sqrt{gk_s s_e}}$.

5. Enter the graph in Figure 6-9 (or Figure 6-10) with the computed values in steps 2 and 3 above and read the value of $\dfrac{V}{\sqrt{\tau/\rho}}$.

6. Compute τ from $\dfrac{V}{\sqrt{\tau/\rho}}$, V and ρ

$$\tau = \frac{V^2\rho}{(V/\sqrt{\tau/\rho})^2}$$

where the terms are defined in the glossary.

Examples - Tractive Stress Approach

Example 6-5

Given: A channel is to be constructed through an area of intense cultivation. The bottom width of the trapezoidal channel is 18 feet with side slopes of 1 1/2:1. The design flow is 262 cfs at a depth of 3.5 feet and a velocity of 3.23 fps. The slope of the energy grade line is 0.0026. There is one curve in the reach, with a radius of 150 feet. The aged n value is estimated to be 0.045. The channel will be excavated in a GM soil that is nonplastic, with D_{75} = 0.90 inches (22.9 mm). The gravel is very angular.

Determine: The actual and allowable tractive stress.

Solution: Since D_{75} > 1/4 inch use the Lane method.

$n_t = (0.90)^{1/6}/39 = 0.0252$ (from Eq. 6-2)

From equation 6-3: $s_t = (n_t/n)^2\ s_e = (0.0252/0.045)^2\ 0.0026 = 0.00082$

actual $\tau_\infty = \gamma_w\ ds_t = (62.4)(3.5)(0.00082) = 0.179$ psf

b/d (ratio of bottom width to depth) = 18/3.5 = 5.14

from Figure 6-3 and 6-4 $\tau_s/\tau_\infty = 0.76$; $\tau_b/\tau_\infty = 0.98$

R_c/b (radius of curve/bottom width) = 150/18 = 8.33

$\tau_{bc}/\tau_b = \tau_{sc}/\tau_s = 1.17$ (Figure 6-5)

Actual $\tau_b = (0.179)(0.98) = 0.175$ psf;

actual $\tau_s = (0.179)(0.76) = 0.136$ psf

Actual $\tau_{bc} = (0.175)(1.17) = 0.205$ psf;

actual $\tau_{sc} = (0.136)(1.17) = 0.159$ psf

Solving for allowable tractive stress -

$\phi_R = 38.4°$ (From Figure 6-7) K = 0.45 (From Figure 6-8)

allowable: $\tau_{Lb} = (0.4)(D_{75}) = (0.4)(0.90) = 0.36$

allowable: $\tau_{Ls} = 0.4\ KD_{75} = (0.4)(0.45)(0.90) = 0.162$

Comparing actual with allowable, the channel will be stable in straight and curved sections.

Example 6-6

Given: A channel is to be constructed through an area of intense cultivation. Bottom width of the trapezoidal section is 18 feet, side slopes are 1-1/2:1. Design flow is 262 cfs, with a depth of 3.5 feet at a velocity of 3.23 fps. Slope of the hydraulic grade line is 0.0026. The design temperature is 50° F. The channel will be cut in nonplastic SM soil, with a D_{75} size of 0.035 inches, a D_{65} size of 0.01075 inches (0.273 mm) and a D_{50} of 0.127 mm. The n value for the channel is 0.045. There are no curves in the reach. Sediment load is quite light in this locality, in the range of clear water criteria.

Determine: The actual tractive stress and the allowable tractive stress.

Solution: Since the D_{75} size is less than 1/4 inch use the reference tractive stress method.

247-927 O - 78 - 12

$\nu = 1.42 \times 10^{-5}$ ft^2/sec., $\rho = 1.940$ lb sec^2/ft^4 (Figure 6-11)

$V^3/g\nu s_e = 3.23^3/((32.2)(1.42 \times 10^{-5})(0.0026)) = 2.83 \times 10^7$

$V/\sqrt{g k_s s_e} = 3.23/\sqrt{(32.2)(0.01075/12)(0.0026)} = 373$

$V/\sqrt{\tau/\rho} = 21.6$ (From Figure 6-9)

$\tau = V^2\ \rho/(V/\sqrt{\tau/\rho})^2 = (3.23^2)\ 1.94/(21.6)^2 = 0.0434$ psf

b/d (ratio of bottom width to depth) = 18/3.5 = 5.14

$\tau_s/\tau \simeq 1.0$; $\tau_b/\tau = 1.31$ (from Figure 6-12 and 6-13)

Actual Tractive Stresses:

$\tau_s = (0.0434)(1.0) = 0.0434$ psf; $\tau_b = (0.0434)(1.31) = 0.0569$ psf

Allowable Tractive Stresses:

$D_{50} = 0.127$ mm; from Figure 6-14 and assuming clear water flow (curve No. 3) the allowable tractive force is 0.025 psf. Both the bed and the banks of the channel are unstable. An evaluation should be made to estimate the magnitude of scour or possible depth of scour before an armor is formed (Refer to next section). Using this procedure in conjunction with the appropriate sediment transport equations, the magnitude of instability can be evaluated.

Formation of Bed Armor in Coarse Material

In material where the coarsest fraction consists of gravel or cobbles an armoring of the bed commonly develops if the allowable tractive stress is exceeded and scour occurs. The depth at which this armor will form may be evaluated if it is determined that some deterioration of the channel can be permitted before stability is reached. The D_{90} - D_{95} size of a representative sample of bed material is frequently found to be the size paving channels when scouring stops. Finer sizes, such as the D_{75} may form the armor, once the finer material is eroded. On the other hand, the coarsest particles may not be sufficiently large to prevent scour. The D_{95} size is considered to be about the maximum for pavement formation within practical limits of planning and design.

The following procedure may be used for determining depth of scour to armor formation.

The actual tractive stress under design hydraulic conditions is computed in accord with equation 6-5. By rearranging this equation

$D = \dfrac{\tau_b}{0.4}$, where D is the limiting size

For example

$\tau_b = 0.6$ psf

Then $D = \frac{0.6}{0.4} = 1.5$ inches.

Reading from the size distribution curve of a representative bed sample, it is determined that 1.5 inches is the D_{90} size. With armor customarily forming as a single layer, the depth of scour to formation of a D_{90} size armor is equal to the D_{90} size in inches divided by the percentage of material equal to or larger than the armor size. For this case $1.5 \div 0.10 = 15$ inches (1.25 ft.) depth to armor formation.

Armoring of the bed will not usually develop initially as a flat bed across the channel. After forming an armor along the thalweg, bars of finer material will next be removed, followed by an increasing attack on the banks.

Tractive Power Approach

General

In general the observations, assumptions, and computational methods used in the development and use of the allowable velocities and the tractive stress methods of analysis are based on correlating a soils erosion resistance with simple index properties determined on disturbed samples. These methods at their present state of development do not assess the effects of cementation, partial lithification, dispersion, and related geologic processes on the erosional resistance of earth materials.

This limitation has been recognized for many years. In the early 1960's, efforts were made by SCS in the Western states to evaluate the stability of channels in cemented and partially lithified soils. The procedures resulting from this effort have come to be known as the Tractive Power Approach.

In this approach the aggregate stability of saturated soils is assessed by use of the unconfined compression test. Field observations of several channels were evaluated against the unconfined compressive strength of soil samples taken from the same channels. The results are shown on Figure 6-15. Soils in channels with unconfined compressive strength versus tractive power that plot above and to the left of the S-line on Figure 6-15 have questionable resistance to erosion. Soils in channels with unconfined compression strength versus tractive power that plot below and to the right of the S-line can be expected to effectively resist the erosive efforts of the stream flow.

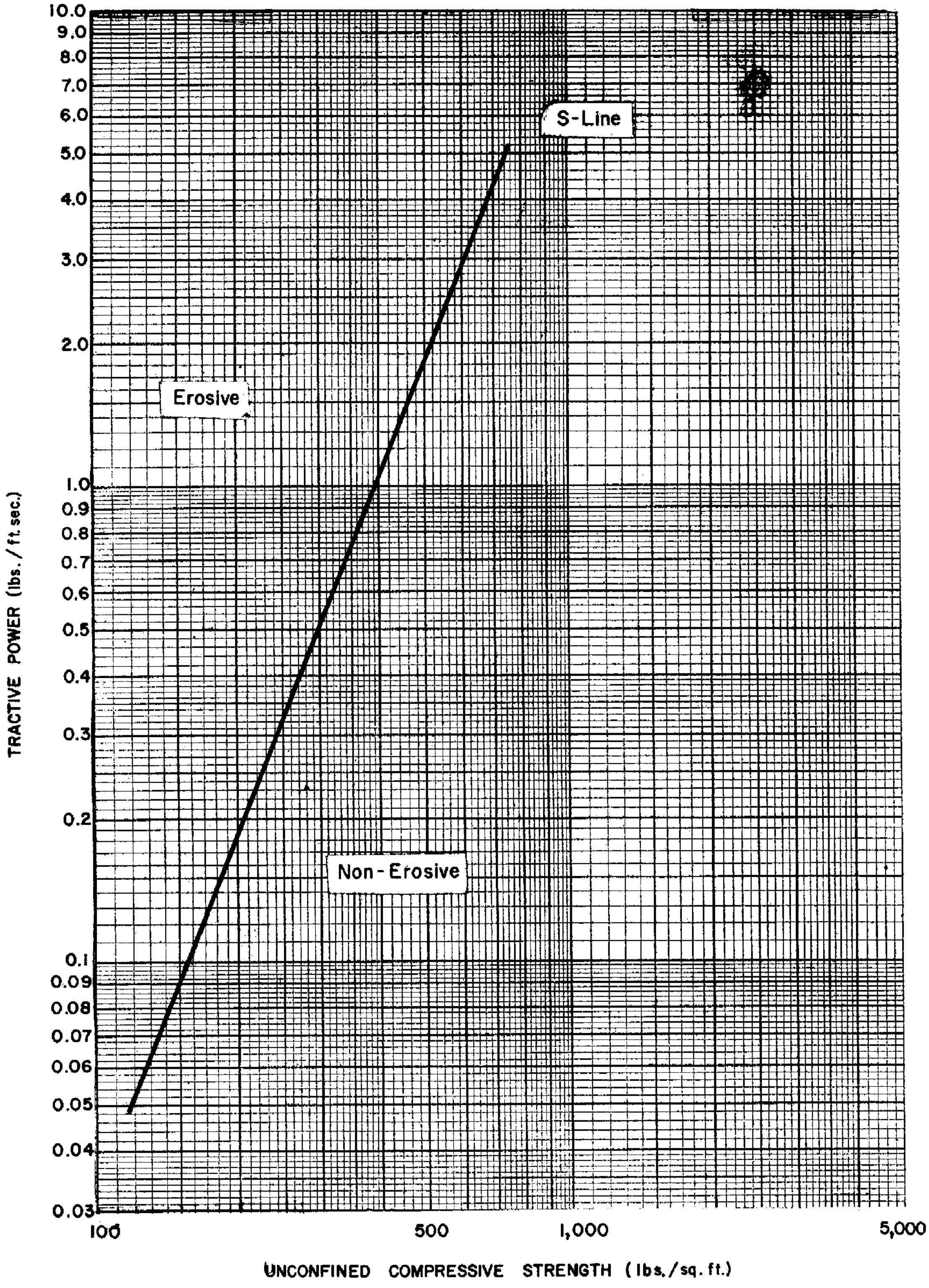


Figure 6-15

Unconfined Compressive Strength And Tractive Power As Related To Channel Stability

Tractive power is defined as the product of mean velocity and tractive stress. Use the appropriate method based on soil characteristics as described in the tractive force procedure to calculate the tractive stress.

<u>Procedure - Tractive Power Approach</u>

The use of tractive power to evaluate earth channel stability involves the following steps:

1. Determine the hydraulics of the channel. This includes hydrologic determinations as well as the stage-discharge relationships for the channel being considered. The procedures to be used in making these determinations are included in chapters 4 and 5 of this Technical Release.

2. Evaluate the sediment transport carrying capacity in the design reach to (a) determine the sediment concentration and (b) test the possibility for aggradation.

3. Determine the physical characteristics, including the saturated unconfined compressive strength of the earth materials in the boundary of the channel. The procedures for making this determination are included in chapter 3 of this Technical Release.

4. Check to see if the Tractive Power Approach is applicable. Use Figure 6-1.

5. Compute the tractive power of the flows being evaluated. Use the mean velocity determined in step one and the procedures in this chapter to determine the tractive stress. Use the method that is appropriate for the grain size of the channel materials.

6. Determine the erosion resistance of the materials in the channel boundary from Figure 6-15.

The following example illustrates the use of the Tractive Power Approach to evaluate channel stability:

<u>Example 6-7</u> - Tractive Power Approach

Given: A channel is to be constructed for the drainage of an area of moderate cultivation. Its bottom width is to be 46 feet, side slopes 2-1/2:1, design flow depth 15.0 feet, and estimated n value of 0.03. The channel will be excavated in clayey silt (ML) having a Plasticity Index of 3 and a D_{75} size of 0.15 mm, a D_{65} size of 0.00256 inches (0.065 mm), and an unconfined compressive strength of 790 psf. The hydraulic gradient is 0.00042, as determined by water surface profile calculations. There are no curves in this reach of channel. The water temperature for the period under consideration is taken as 50°F.

Design flow is 4750 cfs at a depth of 13.5 feet and a velocity of 3.77 fps.

Determine: The actual tractive power and evaluate the stability of the channel.

Solution: Since the D_{75} < 1/4 inch use the reference tractive stress method.

$\nu = 1.42 \times 10^{-5}$ ft^2/sec; $\rho = 1.940$ lb sec^2/ft^4 (Figure 6-11)

$V^3/g\nu s_e = 3.77^3/((32.2)(1.42 \times 10^{-5})(0.00042) = 2.79 \times 10^8$

$V/\sqrt{gk_s s_e} = 3.77/\sqrt{(32.2)(0.00256/12)(0.00042)} = 2220$

From Figure 6-10 $V/\sqrt{\tau/\rho} = 27$

$\tau = V^2 \rho/(V/\sqrt{\tau/\rho})^2 = (3.77^2)\ 1.940/(27)^2 = 0.0378$

$b/d = 46/15 = 3.07$; $\tau_s/\tau = 1.21$ (Figure 6-12)

$\tau_b/\tau = 1.45$ Figure 6-13

$\tau_s = (0.0378)(1.21) = 0.0458$

$\tau_b = (0.0378)(1.45 = 0.0548$

Use the larger of τ_s or τ_b to compute tractive power.

$\tau_b V = (0.0548)(3.77) = 0.207$ - Actual Tractive power

The tractive power versus unconfined compressive strength plots well into the non-erosive zone (Figure 6-15). The channel should be stable for the design conditions.

The Modified Regime Approach

General

The regime theory approach to evaluating channel stability is based on observations of the results in various parts of the world of natural processes causing continuous adjustments of channels. The predictive equations are largely empirical. This method of analysis is limited to flow in alluvial channels.

Chien[49] defines an alluvial channel as one that contains a bed of loose sediment of the same type that is moved along the bed. Such a channel bed seldom remains flat and even. Bars and ripples are developed at the bed surfaces at low stages. They become longer when the discharge increases and eventually may disappear at high flows. At unusually high flows, large, nearly symmetrical, sand bars may appear again, accompanied by surface waves in phase with the bottom undulations. The sand bars and ripples represent another type of roughness, in addition to the roughness of the grains which compose the channel bed. The problem of determining the relationship between slope, depth, velocity, and boundary roughness is complicated by this phenomenon because the roughness not only defines the flow, but the flow itself also molds the roughness.

Blench[50] refers to alluvial channels as those with mobile boundaries. They are the channels that are capable of self-adjustment and have formed their geometric shape by moving boundary material. Materials of at least part of the boundary are moved at some stage of flow. They make at least part of their boundaries from their transported load, and part of their transported load comes from their boundaries.

The equations used in the regime approach have been developed by studying statistics obtained by physical observations of canal systems. Those observations included channel dimension and geometry, and the discharges of streams that were "silt stable," that is, canals that through a succession of years remained free of excessive sediment deposits and did not scour excessively. These equations empirically correlate the capacity of the stream to transport sediment with its main hydraulic characteristics. A "silt stable" or regime stream is known to deposit sediment throughout some stage of flow and to scour during other stages. The proponents of the regime theory approach are satisfied if the net result of deposit and scour is zero at the end of every flow cycle.

When nature or man imposes rigid boundaries in a channel system, the natural laws of alluvial flow are partially or totally negated. Consequently the regime approach cannot be used to analyze rigid boundary channel systems. However, they can be used to determine channel proportions such that the channel can be expected to remain relatively stable.

The procedure and equations presented here are to a large extent from Simons and Albertson.[51]

Three types of mobile boundary materials encompass most, alluvial channels encountered in SCS work. These are: (Note- for this approach a soil is classed as cohesive if the PI is greater than 7.)

Type	Description
A	Sand bed and sand banks
B	Sand bed and cohesive banks
C	Cohesive bed and cohesive banks

A relationship exists between sediment load, Froude number, and channel stability. The Froude number is determined by:

$$F = \frac{V}{\sqrt{g\ d}} \qquad \text{(Eq. 6-12)}$$

According to Simons and Albertson[51/] the Froude number has to be less than 0.3 in type A, B, or C channels to avoid excessive scour.

The relationships between channel geometry and slope are determined by the following regime equations as modified by Simons and Albertson:

$$d = 1.23\ R \quad \text{(For R from 1 to 7)} \qquad \text{(Eq. 6-13)}$$

$$d = 2.11 + 0.934\ R \quad \text{(For R from 7 to 12)} \qquad \text{(Eq. 6-14)}$$

$$W = 0.9\ P \qquad \text{(Eq. 6-15)}$$

$$W = 0.92\ W_T - 2.0 \qquad \text{(Eq. 6-16)}$$

Equations 6-13, 6-14, 6-15, and 6-16 apply to all Type A, B, and C alluvial channels.

Equations 6-17, 6-18, 6-19, 6-20, and 6-21 were derived from data in the Simons and Albertson paper.

Equation	Coefficients by Channel Type			
	A	B	C	
$P = C_1 Q^{0.512}$	3.30	2.51	2.12	(Eq. 6-17)
$R = C_2 Q^{0.361}$	0.37	0.43	0.51	(Eq. 6-18)
$A = C_3 Q^{0.873}$	1.22	1.08	1.08	(Eq. 6-19)
$V = C_4 (R^2S)^{1/3}$	13.9	16.1	16.0	(Eq. 6-20)
$\frac{W}{d} = C_5 Q^{0.151}$	6.5	4.3	3.0	(Eq. 6-21)

where all symbols are as defined in the glossary.

These equations result in some general relationships, approximate width-depth ratios and velocity limitations that if followed will result in "silt stable" channels under the conditions described.

Determination of an acceptable safe slope for a channel is about the most difficult decision in channel design. Values of the slope determined from Manning's equation with a reasonable value of n, cross section geometry consistent with the modified regime equations, and a velocity resulting in a Froude number of less than 0.3 should be compared with the slope determined from equation 6-20.

Procedure - Modified Regime Approach

The use of the modified regime theory to evaluate the stability of earth channels involves the following steps:

1. Determine the hydraulics of the system. This includes hydrologic determinations as well as the stage-discharge relationships for the channel considered. The procedures to be used in this step are included in chapter 4 and chapter 5 of this Technical Release.

2. Determine the character of the earth materials forming the banks and bed of the design reach and the reach upstream.

3. Evaluate the sediment transport carrying capacity in the design reach to (a) determine the sediment concentration and (b) test the possibility for aggradation.

4. Check to see if the modified regime approach is applicable. (Use Figure 6-1)

5. Determine the channel geometry and acceptable safe slope using equations 6-12 through 6-20 with the appropriate constants.

6. Check the slope determined using Manning's equation with a realistic value of n, cross section geometry consistent with that determined in step 5 and a velocity resulting in a Froude number of less than 0.3.

Example 6-8 - Modified Regime Approach

Given: A type B channel to convey 600 cfs at bank full stage. Use 2:1 side slopes and assume that n = 0.022.

Determine: Design the channel.

Solution: Step 1 - Compute $P = 2.51Q^{0.512}$ . . . . Eq. 6-17

$P = (2.51)(600)^{0.512} = 66.4$ ft.

Step 2 - Compute $R = 0.43Q^{0.361}$ . . . . Eq. 6-18

$R = (0.43)(600)^{0.361} = 4.33$ ft.

Step 3 - Compute A = PR or use Eq. 6-19

$A = (66.4)(4.33) = 288$ sq. ft.

Step 4 - Compute $V = Q \div A = 600 \div 288 = 2.08$ fps

Step 5 - Compute d - Since R is less than 7

Use $d = 1.23R$ . . . . Eq. 6-13

$d = 1.23\,(4.33) = 5.3$ ft.

Step 6 - Compute the Froude Number.

$$F = \frac{V}{\sqrt{gd}}$$ . . . . Eq. 6-12

$$= \frac{2.08}{\sqrt{32.2(5.3)}} = 0.159$$

F < 0.3 - Design meets this requirement for stability

Step 7 - Compute bottom width

$W = 0.9\,P$ . . . . Eq. 6-15

$W = 0.9(66.4) = 59.76$ ft.

$W = 0.92\,W_T - 2.0$ . . . . Eq. 6-16

$59.76 = 0.92\ W_T - 2.0$

$W_T = 67.1$ ft.

For 2:1 side slopes-

$b = 67.1 - (4)(5.3) = 45.9$ ft.

Use $b = 45$ ft.

Step 8 - Find the slope of the channel bottom which is needed to cause the channel to be in regime.

$V = C_4\ (R^2 s_o)^{1/3}$ . . . . Eq. 6-20

$2.08 = 16.1\ ((4.33)^2 s_o)^{1/3}$

$s_o = 0.000114$

Step 9 - Find the slope of the channel which is needed to provide capacity assuming uniform flow and Manning's equation. Compute $AR^{2/3}$ using the above values for depth, side slope, and bottom width.

$P = b + 2d\ \sqrt{z^2 + 1} = 45 + 2(5.3)\ \sqrt{5} = 68.7$ ft.

$A = bd + zd^2 = 45\ (5.3) + 2(5.3)^2 = 294.7$ ft.^{2}

$$R = \frac{A}{P} = \frac{294.7}{68.7} = 4.29 \text{ ft.}$$

$AR^{2/3} = (294.7)(4.29)^{2/3} = 778$

Compute s_o

$$Q = \frac{1.486}{n}\ AR^{2/3}\ s_o^{1/2}$$

$$600 = \frac{1.486}{0.022}\ (778) s_o^{1/2}$$

$s_o = 0.00013$

Step 10 - Select slope to be used.

Since two values for the slope have been determined it is necessary to choose a slope that falls between the two values. If a slope flatter than 0.00013 is selected either the width or depth must be increased to provide the needed capacity. In any case the channel will not match the regime relationships exactly but the two slope values are sufficiently close so that the design should be satisfactory. Use the following parameters:

$s_o = 0.00013$

$d = 5.3$ ft.

$b = 45.0$ ft.

Equation 6-21 could have been used to get an idea of what a reasonable width to depth relationship would be from regime methods.

$W/d = 4.3Q^{0.151}$

$W/d = (4.3)(600)^{0.151} = 11.3$

The same ratio would be obtained by dividing the value for W obtained in Step 7 by the depth obtained in Step 5.

$$\frac{W}{d} = \frac{59.76}{5.3} = 11.3$$

Channel Stability With Respect to Sediment Transport

A channel transporting sediment during flow is considered to be stable if the rate of sediment transport is such that the overall equilibrium of the channel is maintained. This requires that scour and aggradation are maintained between prescribed limits. Bedload transport equations have been developed for predicting the rate of transport under equilibrium conditions. In these equations transport is related to stream discharge per foot of channel width. A procedure is presented in this section for determiningn relative rates of scour or deposition using variations in mean velocity.

Application of Bedload Transport Equations

A number of equations have been developed to compute rates of bedload sediment transport. The more widely used include the Einstein Bedload Function[27/], the Meyer-Peter and Muller formula[28/], and the Schoklitsch[29/] equations. A comparison of the measured and computed sediment loads indicates that the most reliable involves depth-integrated samples of suspended load and computations employing the Einstein bedload function. This is known as the

Modified Einstein Procedure[30]. However, the field data required in use of this procedure are not ordinarily available.

The Einstein bedload function and the Meyer-Peter and Muller formula for computing bedload transport have been determined to be about equally adapted for this purpose in the range from medium-size sand to gravel. The equations for computation of equilibrium bedload transport are given in the references cited.

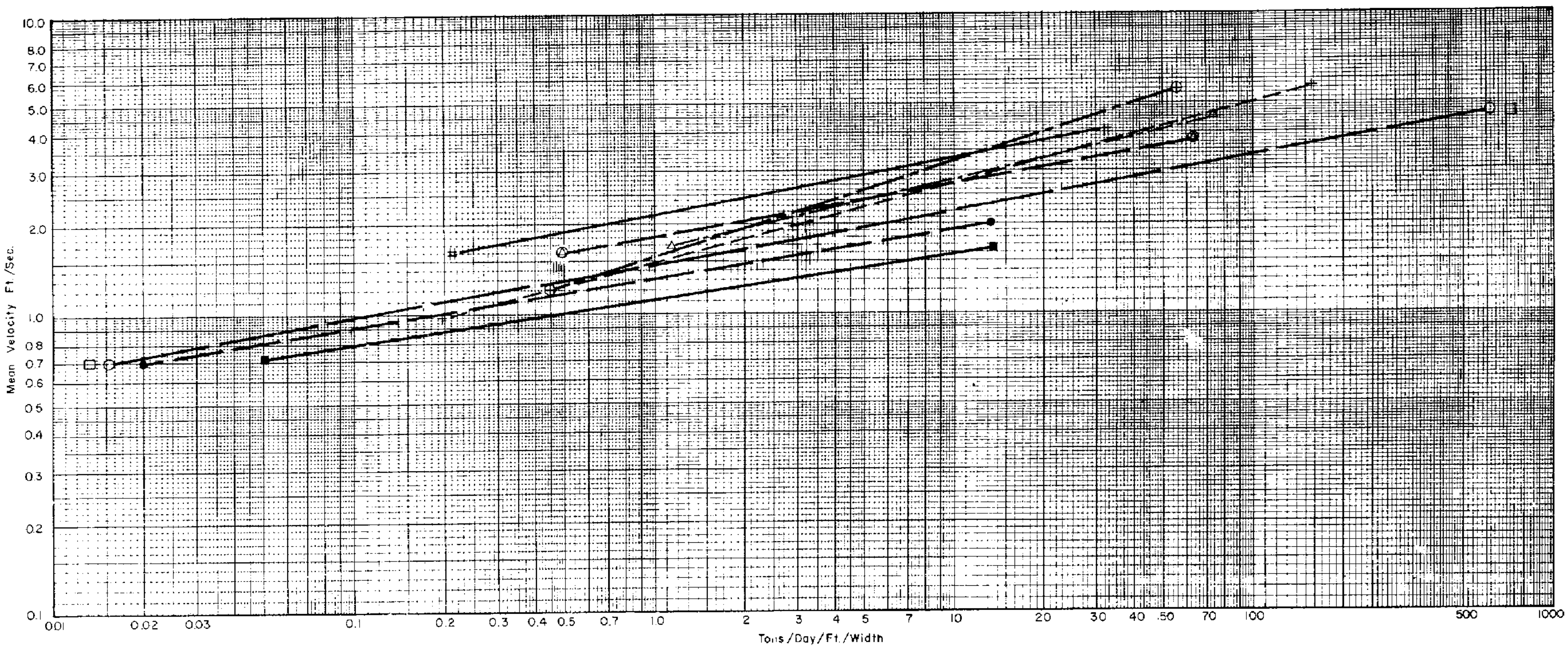
Sediment Transport in Sand Bed Streams Not in Equilibrium

The procedures described in this section are recommended for determining the effect of channel changes on stability. They are based on research which shows that the rate of bedload sediment transport is strongly related to mean velocity. Figure 6-16 shows this relationship for fine and medium sand sizes. Factors which create differences in mean velocity from one reach to another cause differences in rates of bedload transport. If the changes in rates are substantial in amount and in duration, an unstable condition is established.

Bedload sediment transport in sand bed streams with variable roughness. Numerous studies have indicated that the roughness coefficient n varies in a sandbed stream as the bed form changes in response to the formation of ripples, dunes and anti-dunes. No generally accepted method has been developed for predicting what the n value will be at any given discharge or velocity. In the approach used here, mean velocity is related to an approximation of tractive force for broad, shallow flow - - the product of depth, slope and unit weight of water. The relationship is established by adaptation of data presented by Dawdy[31]. In his paper, the hydraulic radii as related to mean velocity are shown for a number of sand bed streams. Figure 6-17 shows a plotting of mean velocity related to the product of slope, hydraulic radius and unit weight of water for five of the streams. It is assumed that hydraulic radii in these relatively broad, flat-bedded streams are equivalent to depth for purposes of computing tractive force in this procedure. More data are needed to define the curve of Figure 6-17 for sediment with the median size coarser than 0.5 mm. Data from one stream with a median sand size of 0.8 mm. indicates a deviation from this curve.

Procedure- Channel Stability with respect to Sediment Transport

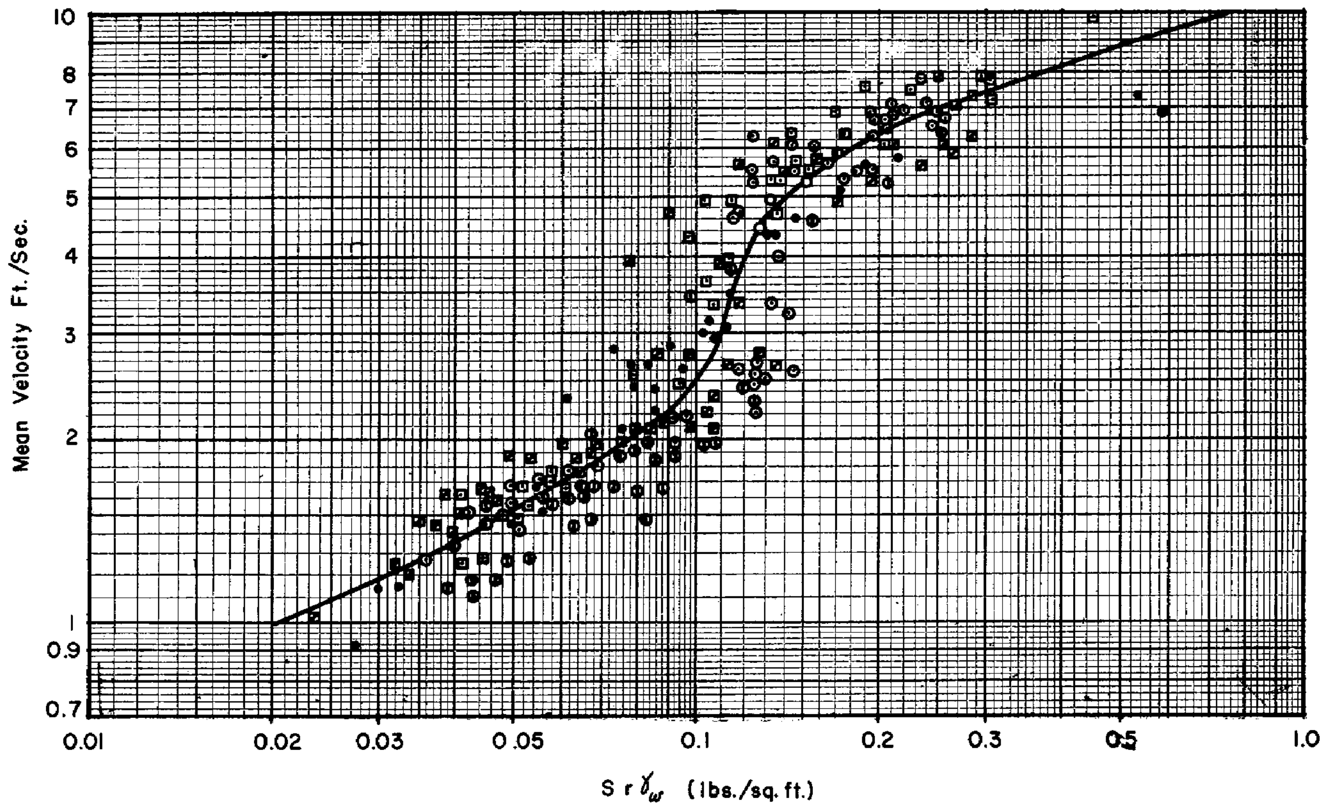
The following procedure may be used to determine whether unstable conditions will occur under projected channel conditions where variable bed roughness occurs:



NO.	D_{50} SIZE	MEASURING POINT	WATER DEPTH	REFERENCE
1	○ 0.45 mm	SAND-FLUME	0.27'- 1.0'	U.S.G.S. W.S. PAPER 1468-A 61
2	△ 0.594 mm	SAND-FLUME	0.074'-0.346'	CAL. TECH. REPORT NO. K.H.R. 2 JAN. 61
3	⊘ 0.233 mm	SAND-FLUME	0.145'-0.356'	CAL. TECH. REPORT NO. K.H.R. 2 JAN. 61
4	□ 0.145 mm	SAND-FLUME	0.240'-0.242'	CAL. TECH. REPORT NO. E-68 DEC. 57
5	■ 0.137 mm	SAND-FLUME	0.203'-0.302'	CAL. TECH. REPORT NO. E-68 DEC. 57
6	● 0.137 mm	SAND-FLUME	0.524'-0.553'	CAL. TECH. REPORT NO. E-68 DEC. 57
7	+ 0.130 mm	RIO GRANDE AT BERNALILLO		U.S.B.R. - SED. CONF. PAPER NO. 32 DEC. 63
8	# 0.27 mm	COLORADO R. AT ADOBE		U.S.B.R. - SED. CONF. PAPER NO. 32 DEC. 63
9	⊕	AVE. CURVE - FIVE MILE CR. WY. NIOBRARA R. MIDDLE LOUP, NEB.		TRANS. AGU. VOL. 38 NO. 5 OCT. 57

Figure 6-16

RELATIONSHIP BETWEEN MEAN VELOCITY AND SEDIMENT TRANSPORT ON AND NEAR STREAM BED



◪	0.40 mm	SAND	PIGEON ROOST CR., MISS.
⊕	0.32 mm	SAND	REPUBLICAN R., NEB.
⊡	0.26 mm	SAND	MIDDLE LOUP R., NEB.
⊙	0.30 mm	SAND	RIO GRANDE R., N.M.
•	0.50 mm	SAND	SOUTH FK. POWDER R., WYO.

FIGURE 6-17

RELATION OF MEAN VELOCITY TO PRODUCT OF SLOPE
HYRAULIC RADIUS AND UNIT WEIGHT OF WATER

FINE AND MEDIUM SIZE SAND BED STREAMS DATA ADOPTED FROM
U.S. GEOL. SURVEY WATER SUPPLY PAPER 1498 C, FIG. 5, 6, 8, 9, 10

1. Determine whether a full supply of bedload will be introduced into the reach by methods described in Chapter 3, NEH Section 3 - Chapter 4 and Geologic Note 2.

2. Compute mean velocities for various stages of flow for a hydrograph or series of hydrographs at cross sections typical of stream reaches to be compared. The recommended method of determining the influence of variable bed roughness and bank roughness on mean velocity is explained in the example of the procedure given below.

3. Select one characteristic velocity-bedload transport curve from Figure 6-16 or construct a new one from available data.

4. Compute rates of bedload transport for each reach.

5. Where scour or aggradation may occur, revise design, such as changing projected channel slope, width and depth. The design may have to provide sufficient channel freeboard for low flow aggradation to insure capacity during large flows. Reliance on the removal of the low flow deposits prior to peaking of higher flows should be approached with caution since the sequence of flows or the condition of the channel at the time of any flood occurrence cannot be predicted.

Example 6-9

Assume that a flood detention reservoir will be built on a sand bed stream with a median size of bed material of 0.15 mm. With the reservoir installed, improvement of the channel two miles downstream will be required to allow controlled runoff without erosion of the banks. The distance from the dam to the beginning of the reach to be improved is great enough to enable the flow to become fully loaded with bed material. The energy gradient in the unimproved reach is 0.003 feet per foot. The stream banks are nearly vertical and free of vegetation.

Rights-of-way limitations show that a 60-foot bottom width improved channel meets requirements. It is proposed to protect erodible banks with riprap on a slope of 2-1/2:1. The energy gradient in this reach is computed to be 0.0025.

The riprapped slopes and the narrower and lower gradient section result in a change in velocity over that of the upstream section. Table 6-1 and its supplement, Table 6-2, show the procedure used in computing these velocities. The formula in Table 6-2 was originally presented by Horton[32]. The computation of the values in each of the columns is as follows.

Column 1 - Depths are chosen to provide a range of flows within the two channel sections up to the maximum proposed reservoir release rate.

Columns 2 and 3 (80-foot section), and Columns 6 and 7 (60-foot section) give the cross-sectional area at the specified depths and side slopes. These data were obtained from hydraulic tables such as those prepared by the Corps of Engineers.

Columns 4 and 8 are approximations of tractive force using the energy gradients equal to 0.003 or 0.0025, respectively; depth (column 1), and the unit weight of water, 62.4 pounds per cubic foot.

Columns 5 and 9 are obtained from Figure 6-17, the mean velocities being read from the intersection of the product values in columns 4 and 8 with the curve.

The remaining calculations determine the correction of velocity of the 60-foot section due to riprap. Velocities that are attributable to the depth, energy gradient and bed roughness only are reflected in the velocities in Columns 5 and 9. Column 10 shows the n values related to the velocities in Column 9. In this example the n for the velocities in Column 9 were obtained from the Table of Values of nv Corresponding to Different Values of R (radius) and s (slope) in Manning's Formula in Kings "Handbook of Hydraulics."[33/] These n values are used in the formula given at the head of Table 6-2 for computing roughness of the improved channel, accounting for both riprap and bottom roughness.

The method of obtaining the n in Column 11, corrected for roughness due to the riprapped side slopes, is given in Table 6-2.

The corrected mean velocity of Column 12 is determined from Manning's formula, using corrected n of Column 11, an energy gradient of 0.0025 and the appropriate R in Column 6.

The available reservoir storage capacity and the hydrology of the site indicates that 500 cfs is the maximum desired capacity of the principal spillway. Figure 6-18 gives the design release rate for the proposed reservoir. It is assumed that there are no significant uncontrolled flows entering the stream between the reservoir and improved channel.

In design of the improved channel and in programming reservoir releases, it is necessary to determine (1) if the proposed reservoir releases will provide equilibrium bedload transport through the improved channel; and (2) if scour or aggradation will occur and the relative rate of its occurence.

The foregoing calculations have provided data on velocities for a range of depths through the reaches represented by the two channel sections. The following steps are necessary to determine how the changes in velocity for the same discharge passing through the two reaches affect their capacity to transport bedload sediment.

Velocity-area curves for the two stream reaches were prepared from the data in Columns 3 and 5 (for the 80-foot wide channel), and Columns 7 and 12 (60-foot wide channel). The curves in Figure 6-19 provide information that enables calculations and plotting of discharges as related to velocities. In the velocity-discharge curves, Figure 6-20 discharges

247-927 O - 78 - 13

Table 6-1 - Mean Velocity Computations - Two Channel Sections

Channel - 80' bottom width Energy gradient - 0.003 ft./foot 1/4:1 Side Slopes					Channel - 60' bottom width Energy gradient - 0.0025 ft./foot 2 1/2:1 Side Slopes						
1	2	3	4	5	6	7	8	9	10	11	12
										Corrected for Riprap	
Depth Ft.	Hyd. Radius Ft.	Area Sq. Ft.	Tractive Force Lbs/sq.ft.	Mean Veloc. fps	Hyd. Radius Ft.	Area Sq. Ft.	Tractive Force Lbs/sq.ft.	Mean Veloc. fps	"n" Related to Velocity	"n"	Mean Velocity fps
0.2	0.2	16	0.037	1.3	0.2	12.1	0.031	1.2	0.0212	0.0220	1.15
0.4	0.4	32	0.075	1.8	0.39	24.4	0.062	1.7	0.0233	0.0240	1.65
0.6	0.59	48	0.112	3.4	0.58	36.9	0.094	2.3	0.0225	0.0231	2.24
0.8	0.79	64.2	0.150	5.2	0.77	49.6	0.125	4.4	0.0142	0.0170	3.68
1.0	0.98	80.25	0.187	6.1	0.96	62.5	0.156	5.4	0.0134	0.0160	4.45
1.4	1.36	112.25	0.262	7.1	1.32	88.9	0.218	6.6	0.0135	0.0165	5.42
1.6	1.54	128.6	0.300	7.4	1.49	102.4	0.250	7.0	0.0137	0.0172	5.64
2.0	1.91	161.0	0.374	8.0	1.84	130.0	0.312	7.4	0.0151	0.0185	6.03
2.6	2.46	209.7	0.487	8.8	2.34	172.9	0.406	8.2	0.0160	0.0202	6.47
3.0	2.81	242.2	0.562	9.2	2.66	202.5	0.468	8.5	0.0168	0.0213	6.69
3.6	3.33	291.2	0.674	9.7	3.13	248.4	0.562	9.2	0.0173	0.0221	7.18
4.0	3.67	324.0	0.749	10.0	3.43	280.0	0.624	9.5	0.0178	0.0230	7.34

Table 6-2 - Calculation of n adjusted for 2 1/2 to 1
side slopes (riprapped)

The formula used for obtaining the corrected n in Column 11 is:

$$n = \left(\frac{P_1 \, n_1^{3/2} + P_2 \, n_2^{3/2}}{\underline{P}} \right)^{2/3}$$

where n_1 = roughness coefficient of the individual lining material

n_2 = roughness coefficient of riprapped banks

P_1 = wetted perimeter associated with roughness coefficient n (bottom width)

d = depth

z = slope of banks

$P_2 = (1 + z^2)^{1/2}$ (2d) (wetted perimeter of the banks)

$\underline{P}$ = total wetted perimeter ($P_1 + P_2$ etc.)

1	2	3	4	5	6	7	8	9	10	11
P_1	n_1	$n_1^{3/2}$	d	z	$(1+z^2)^{1/2}$	P_2	n_2	$n_2^{3/2}$	$\underline{P}$	Corrected n
60	0.0212	0.0030	0.2	2.5	2.692	1.077	0.035	0.0065	61.077	0.0220
"	0.0233	0.0036	0.4	"	"	2.154	"	"	62.154	0.0240
"	0.0225	0.0034	0.6	"	"	3.230	"	"	63.230	0.0231
"	0.0142	0.0017	0.8	"	"	4.307	"	"	64.307	0.0170
"	0.0134	0.0016	1.0	"	"	5.384	"	"	65.384	0.0160
"	0.0135	0.0016	1.4	"	"	7.540	"	"	67.540	0.0165
"	0.0137	0.0016	1.6	"	"	8.614	"	"	68.614	0.0172
"	0.0151	0.0018	2.0	"	"	10.770	"	"	70.770	0.0185
"	0.0160	0.0020	2.6	"	"	14.00	"	"	74.00	0.0202
"	0.0168	0.0022	3.0	"	"	16.15	"	"	76.15	0.0213
"	0.0173	0.0023	3.6	"	"	19.38	"	"	79.38	0.0221
"	0.0178	0.0024	4.0	"	"	21.54	"	"	81.54	0.0230

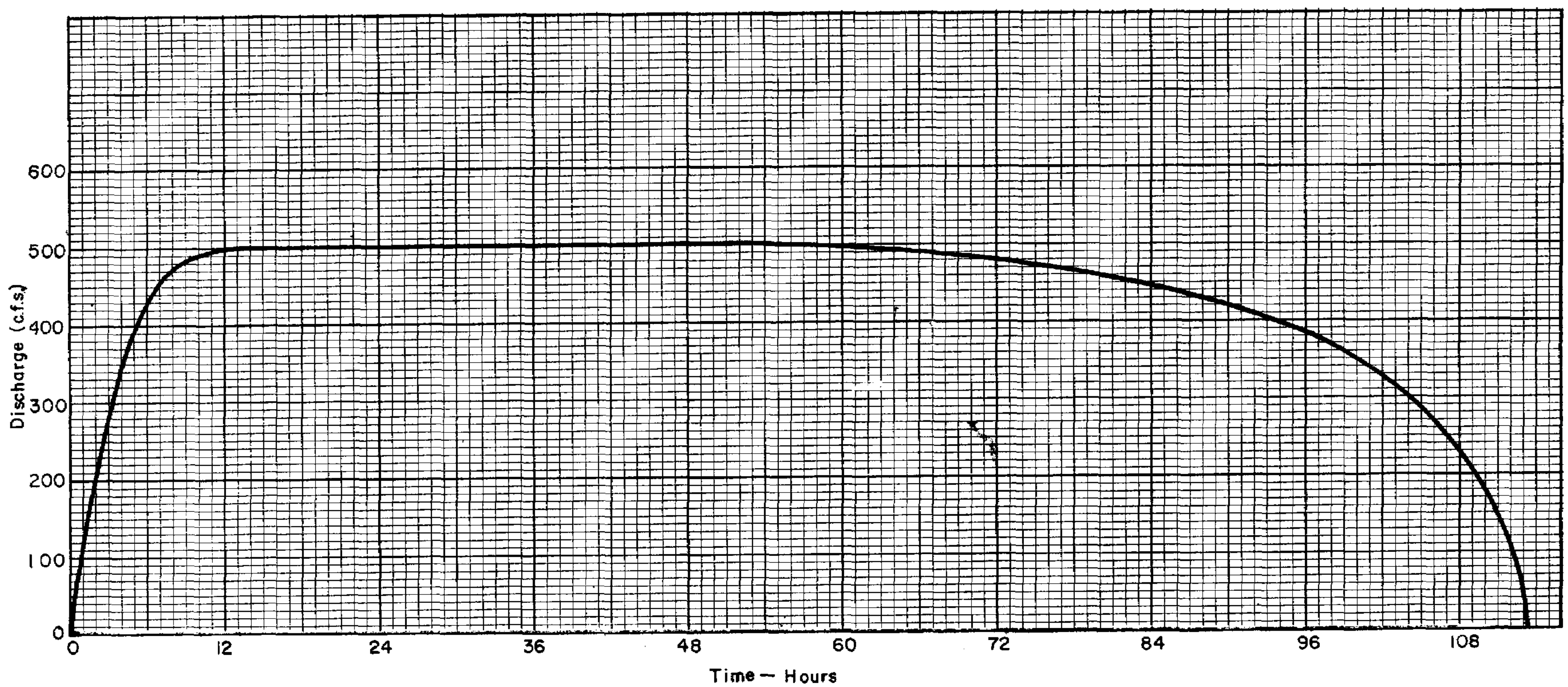


FIGURE 6-18
RESERVOIR RELEASE HYDROGRAPH
PRINCIPAL SPILLWAY

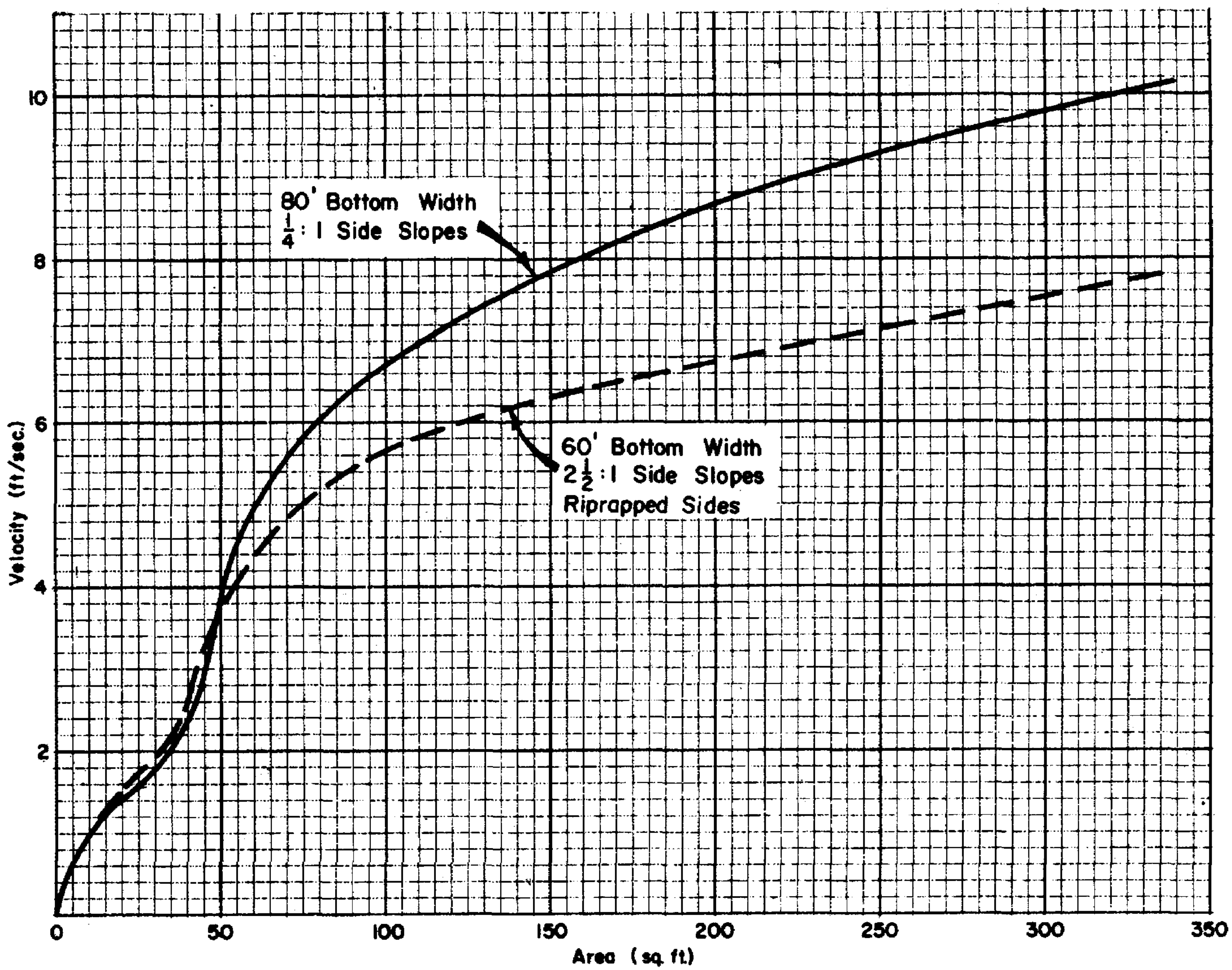


FIGURE 6-19
VELOCITY - AREA CURVE

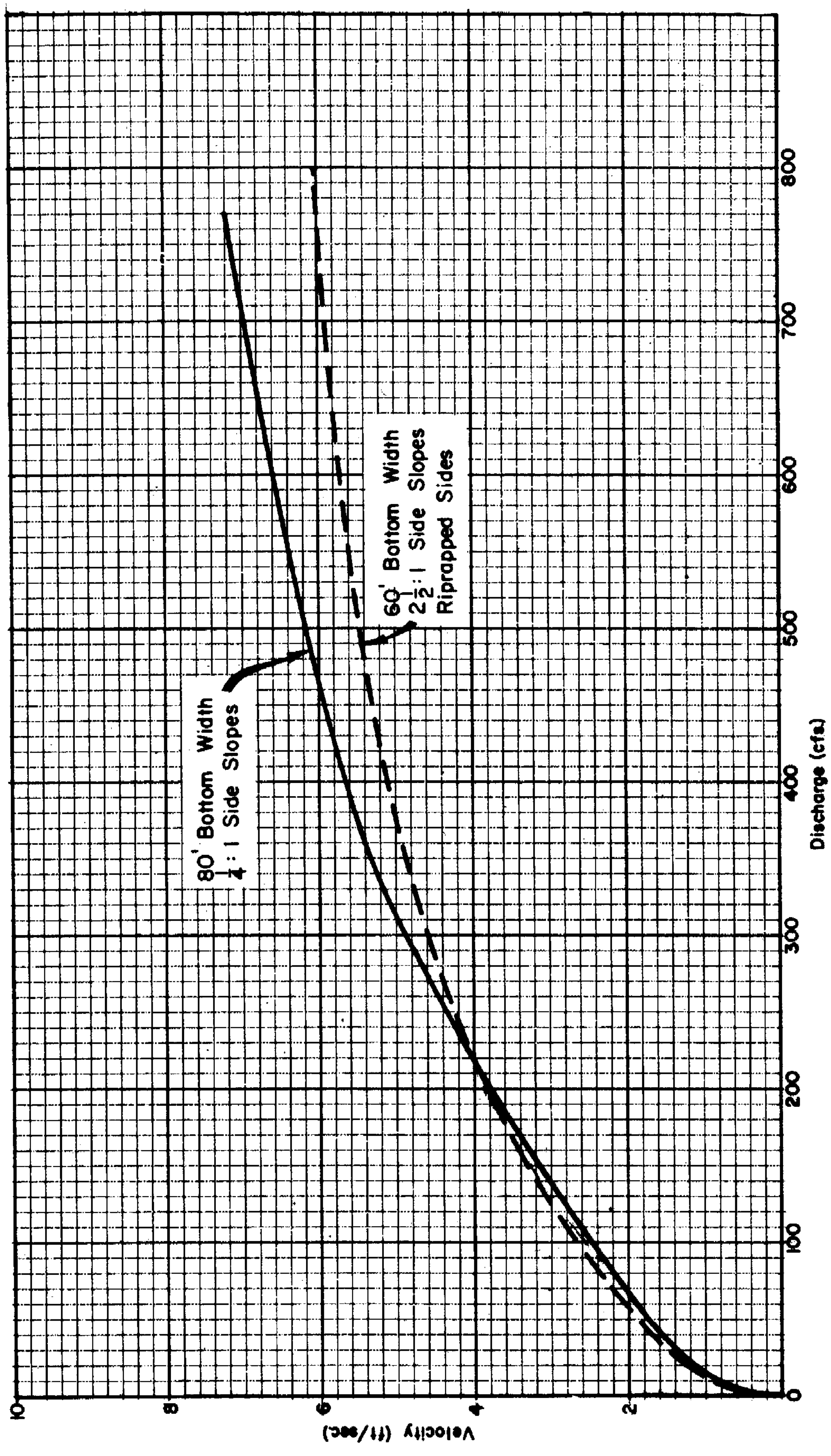


FIGURE 6-20
VELOCITY-DISCHARGE CURVE

were obtained for each 0.5 fps increase in velocity from the product of the cross-sectional area and equivalent velocity for the two stream sections shown on Figure 6-20.

The next step required selection of a mean velocity-bedload sediment transport rating curve from Figure 6-16 because of similarity in median grain size. Curve number 7 was chosen as the more applicable to this problem. Selection of another curve would result in relatively comparable qualitative results but they would differ quantitatively. However, qualitative results provide useful information for solution of this problem.

The discharge-bedload sediment transport curves of Figure 6-21 were derived from Figure 6-16 and Figure 6-20, in the following manner: Velocities for the same discharges in both stream reaches were read from the curve of Figure 6-20. In the example, the discharges selected were spaced sufficiently close (every 50-75 cfs. change) to provide adequate plotting points for drawing a curve. The sediment transport for the velocities relating to the discharges were read from Curve 7, Figure 6-16. The resulting data enabled plotting of the discharge-bedload sediment transport curves of Figure 6-21.

The remainder of the steps in this problem are indicated in Table 6-3, which shows bedload sediment transport determinations for the 80-foot bottom width and 60-foot bottom width stream reaches. Column 1 gives the range in discharge for a number of segments of the reservoir release hydrograph of Figure 6-18. The segments are so selected as to facilitate location of a point (Column 2) reflecting a mean value for the range. The elapsed time covered on the hydrograph by the discharge range in Column 1 is given in Column 3. Since sediment transport is in tons per day per foot of width on Figure 6-16 and Figure 6-21 the elapsed time is converted to percent of 24 hours in Column 4. Bedload sediment transport in tons per day per foot of width in Column 5 is the intersection of the mid-point value in Column 2 with the curve on Figure 6-21 for the appropriate channel reach. Bedload sediment transport is the product of the data in Columns 4 and 5 and the bottom width of the respective reach.

The results of the procedure applied to the example show that equilibrium transport could be maintained if the maximum reservoir release were about 150 cfs. Beyond that discharge, the improved section would aggrade with about 57 percent of the incoming bedload sediment moving through. At a maximum release rate of 500 cfs. aggradation could soon fill the channel, depending on the frequency of reservoir release and length of reach over which the deposit would accumulate. Presuming the release rate could not be reduced to 150 cfs., a great reduction in aggradation could be achieved by a change to about 300 cfs. maximum reservoir release. This is evident from the data on Table 6-3 and the increased difference in transport between the two reaches at the higher release rates.

Table 6-3
Bedload Sediment Transport
80-foot channel, energy gradient 0.003 ft./ft., 1/4:1 side slopes

1	2	3	4	5	6
Range In Discharge cfs	Mid-Point of Range cfs	Elapsed Time Hrs.	Time % of 24 Hrs.	Sediment Transport per ft.width tons/day	Bedload Sediment Transport Col. 4 x 5 x Bottom Width Tons
0 - 200	100	1	4.2	7	24
200 - 400	285	4	9.6	78	599
400 - 500	475	7	29.1	200	4,656
500 - 500	500	30	124.7	213	21,248
500 - 485	492	24	100	210	16,800
485 - 460	474	12	50	200	8,000
460 - 418	441	12	50	180	7,200
418 - 327	380	12	50	142	6,480
327 - 240	290	6	25	82	1,640
240 - 0	155	6	25	18	720
		Total bedload sediment transport			67,367

60-foot channel, energy gradient 0.0025 ft./ft., 2-1/2:1 side slopes

0 - 200	100	1	4.2	9	43
200 - 400	285	4	9.6	67	386
400 - 500	475	7	29.1	140	2,450
500 - 500	500	30	124.7	145	14,460
500 - 485	492	24	100	142	8,510
485 - 460	474	12	50	139	4,170
460 - 418	441	12	50	130	3,900
418 - 327	380	12	50	106	3,180
327 - 240	290	6	25	69	1,035
240 - 0	155	6	25	21	315
		Total bedload sediment transport			38,449

The effect of widening the improved reach to that of the upstream reach would need to be re-evaluated because the n attributable to the channel would be changed. Determination of the most efficient channel dimension and reservoir release commensurate with the site limitation may require several trial and error computations.

Bedload sediment transport in sand bed streams with constant roughness. - - The following procedure may be used in sand bed streams with a median size larger than 0.5 mm. and with constant bed roughness. The steps to be taken are the same as those given in the example 6-9 except for the variable roughness computations.

Example 6-10

Assume that stabilization requires construction of a concrete-lined channel from the edge of the foothills of a tributary across the floodplain to its junction with the main stream. The natural channel within the foothills contains a full supply of coarse sand with a median size of 1.0 mm. The width of the stream averages 28 feet and has a gradient of 0.0195 feet per foot. The slope of an alluvial fan just downstream from the foothill zone is 0.014 feet per foot. The tributary joins another tributary which has a grade of 0.006 feet per foot at the junction.

The hydrology and hydraulics of the proposed improvements show that an 8 foot wide rectangular section will be required to handle the tributary flow on a grade of 0.014 feet per foot, whereas a rectangular section 14 feet wide will be required below the junction of the two streams. Determination of relative rates of bedload transport in the natural channel, tributary section and at the junction of the tributaries is necessary to predict whether the lined sections will carry the introduced bedload or whether a debris basin may be required.

The estimated constant n for the 3 stream sections is 0.027 for the 28' bottom-width sand bed stream above the concrete lining and 0.014 for the lined sections.

Figure 6-22 shows a synthetic hydrograph for a relatively frequent event and Table 6-4 presents the mean velocity and discharge for stages of flow that would be experienced during the runoff. These correspond to Figure 6-18 and Table 6-1 in the Procedural Guide. The derivation of Figure 6-23 (Velocity-Area Curve) and Figure 6-24 (Velocity-Discharge Curve) are described for their counterparts, Figures 6-19 and 6-20 in the Guide. Curve 9, Figure 16 of the latter was extended beyond the data range in order to estimate sediment transport at higher velocities. Rates of sediment transport per foot of width on Figure 6-18 for the 3 described channel sections is equivalent to Figure 6-21 of Guide.

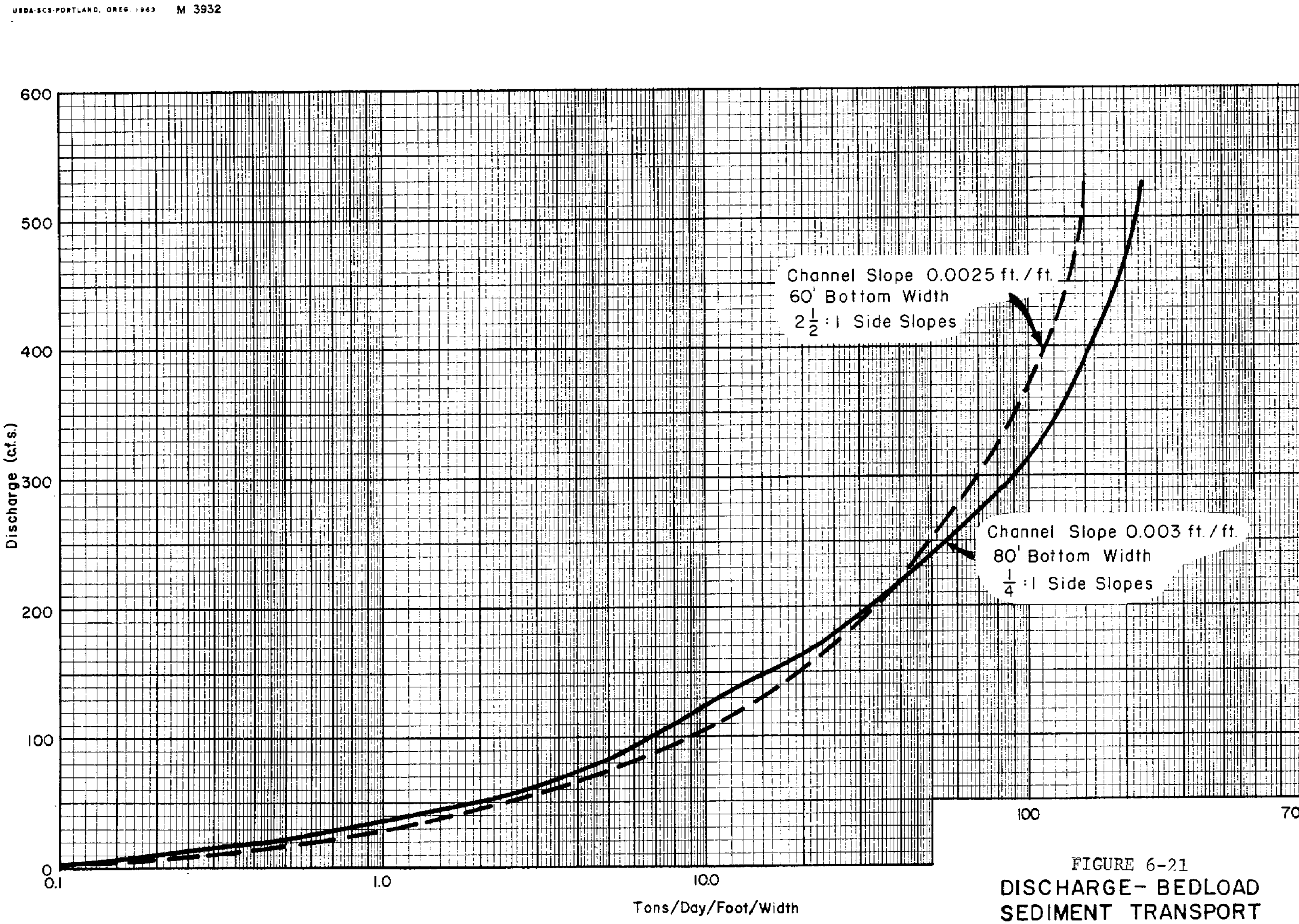


FIGURE 6-21
DISCHARGE- BEDLOAD
SEDIMENT TRANSPORT

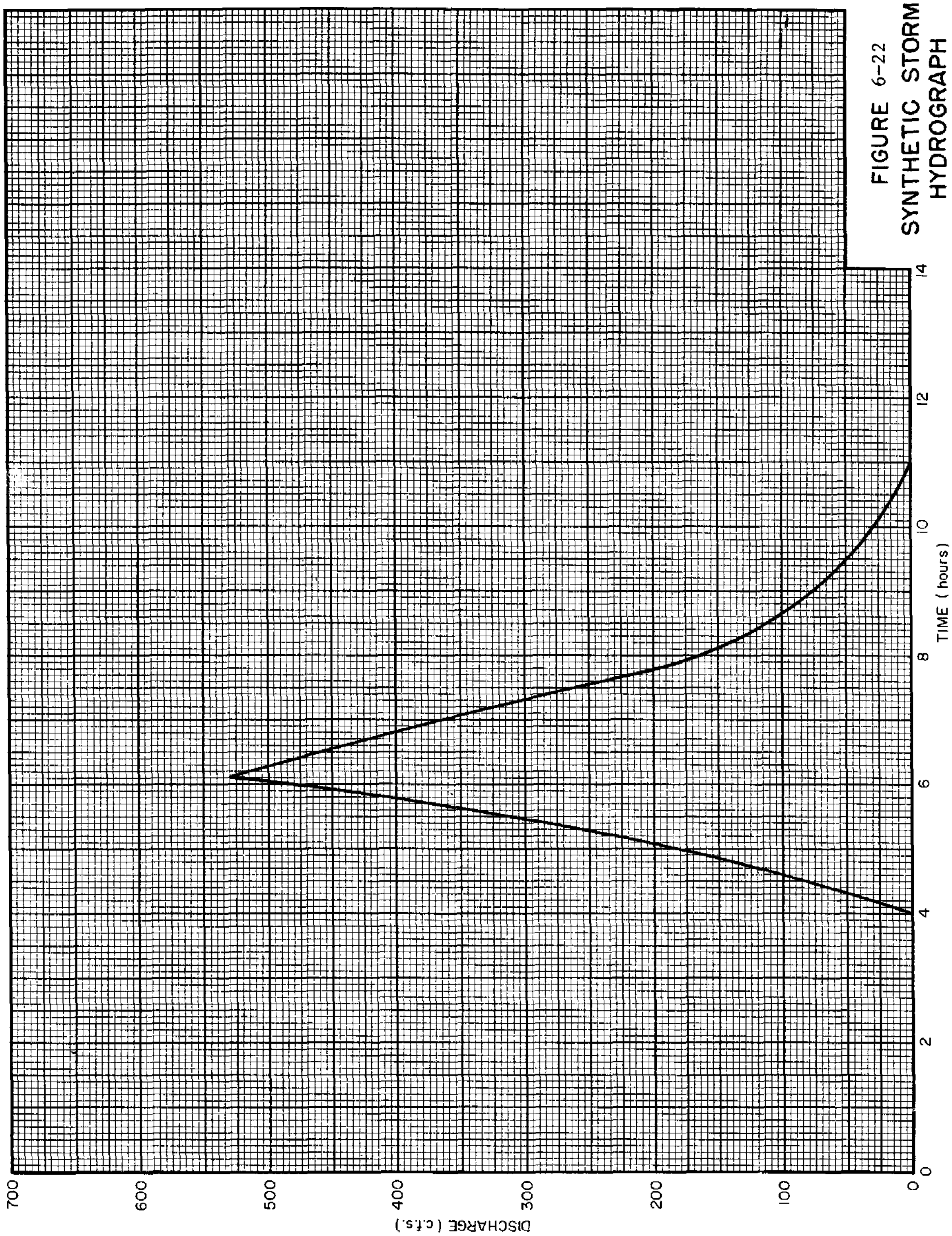


FIGURE 6-22
SYNTHETIC STORM
HYDROGRAPH

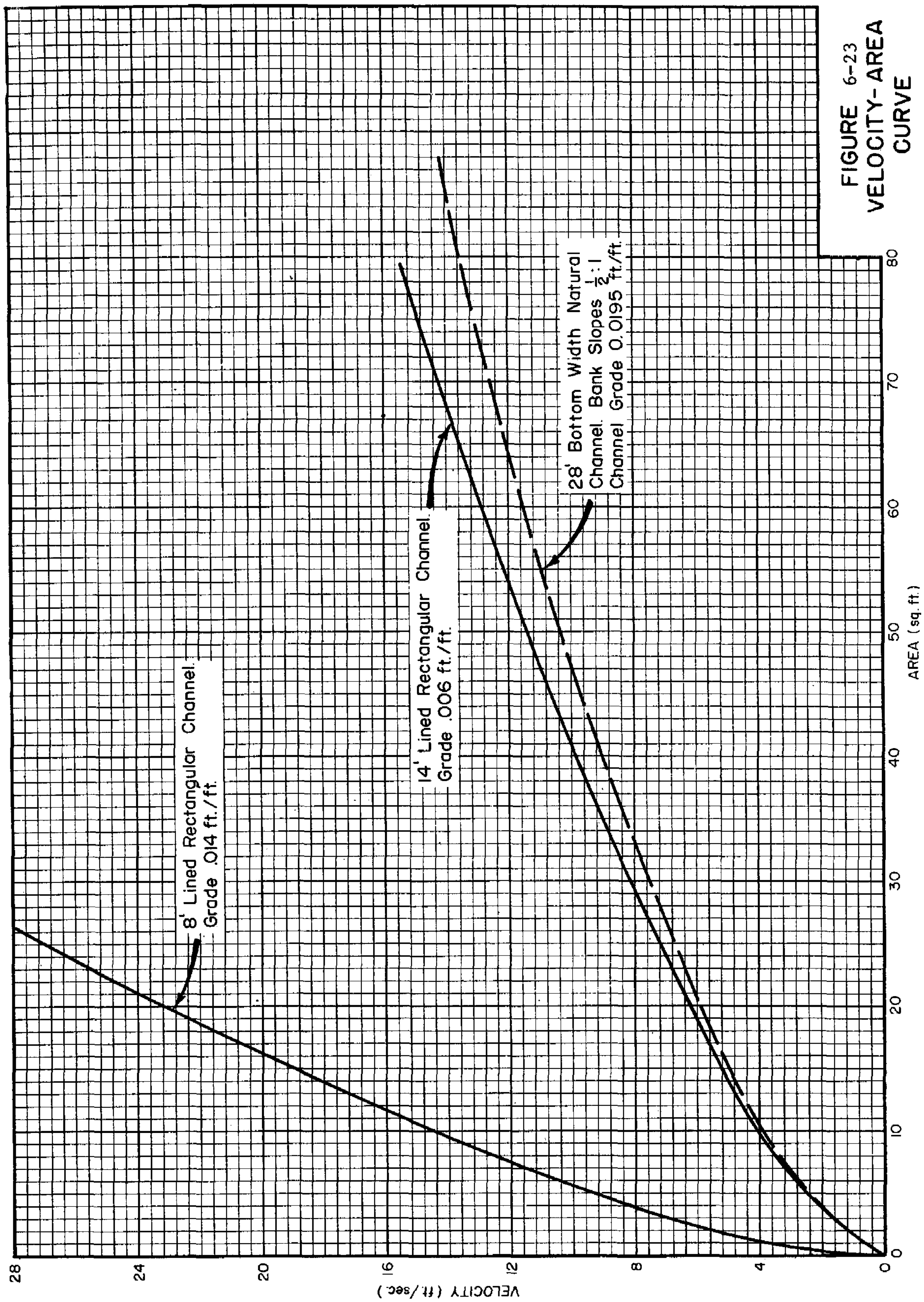


FIGURE 6-23
VELOCITY-AREA CURVE

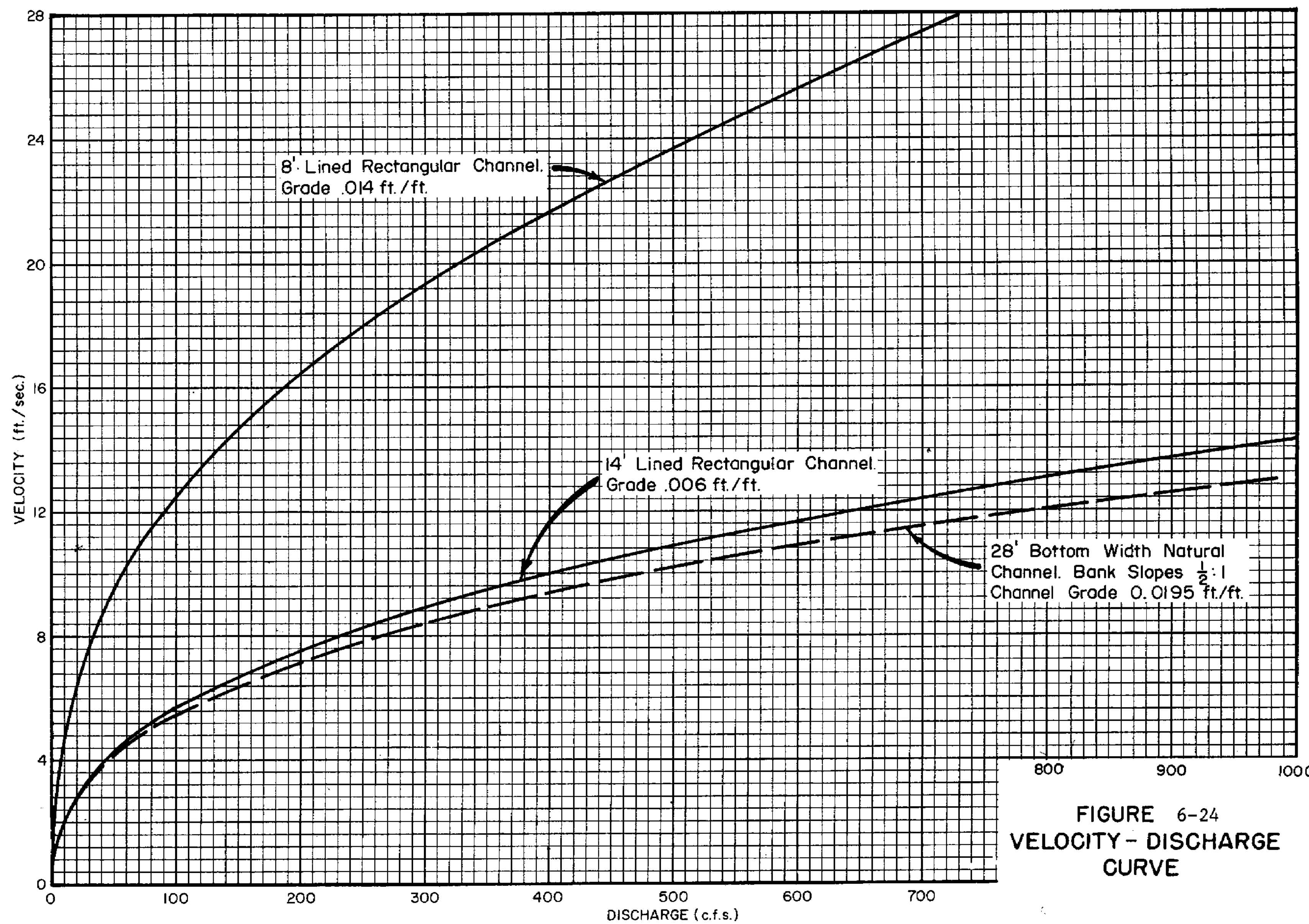


FIGURE 6-24
VELOCITY – DISCHARGE CURVE

Table 6-4 - Mean Velocity Computations - Three Channel Sections

Natural Channel - 28' bottom width Slope - 0.0195 ft./ft. 1/2:1 Side Slopes n -0.027					Channel 8' Rectangular Slope -0.014 ft./ft. n -0.014			Lined Rectangular Channel 14' width Slope -0.006 ft./ft. n -0.014		
1	2	3	4	5	6	7	8	9	10	11
Depth	Hyd. Radius	Area	Mean Velocity	Discharge	Area	Mean Velocity	Discharge	Area	Velocity	Discharge
ft.	ft.	sq.ft.	fps	cfs	sq.ft.	fps	cfs	sq.ft.	fps	cfs
0.2	0.20	5.62	2.61	15	1.6	4.30	7	2.8	1.67	5
0.4	0.39	11.28	4.09	46	3.2	6.80	22	5.6	2.65	15
0.6	0.58	16.98	5.29	90	4.8	8.92	43	8.4	3.48	35
0.8	0.76	22.72	6.38	145	6.4	10.84	69	10.2	4.21	43
1.0	0.94	28.50	7.37	210	8.0	12.57	101	14.0	4.89	68
1.2	1.12	34.32	8.23	282	9.6	14.39	138	16.8	5.53	93
1.4	1.29	40.18	9.10	366	11.2	15.70	175	19.6	6.13	120
1.6	1.46	46.08	9.79	451	12.8	17.15	220	22.4	6.67	149
1.8	1.62	52.02	10.55	550	14.4	18.50	266	25.2	7.22	182
2.0					16.0	19.90	318	28.0	7.76	218
2.2					17.6	21.2	373	30.8	8.28	255
2.4					19.2	22.4	430			
2.6								36.6	9.25	338
2.8					22.4	24.8	555			
3.0								42.0	10.2	429
3.4								47.6	11.1	529

Table 6-5 gives the results of Bedload Sediment Transport calculations in the 3 sections. The results show that the steeper concrete lined section between the tributary junction and the natural channel can carry more than 3 times the bedload sediment introduced by the storm. However, the channel below the junction can carry little more than half the amount introduced from the natural channel. The result would be a plugging at the junction and backfilling of sediment into the contributing channel. About 50 percent additional inflow of relatively sediment free water from the other tributary would be necessary for prevention of a plug forming at the junction. A basic reason for this problem developing is indicated by inspection of Figure 6-25. The sediment transport curves for the 14' lined channel at grade of 0.006 feet per foot shows rates of transport slightly in excess of that for the 28' incoming channel section per foot width. Since the latter is twice as wide, transport over the whole width substantially exceeds that of the lined section. The need for construction of a debris basin to trap the bedload sediment is indicated in this example.

Slope (Bank) Stability Analysis

General

A slope stability analysis from the standpoint of strength will not be required for all channel banks - - in fact, it may not be required for the majority of channels. However, soil conditions and all of the forces that may act on a slope should be carefully considered in deciding whether or not an analysis is needed. In some cases, it may not be economically feasible to initially construct channel banks in accordance with design requirements imposed by high water table, quicksand or "soupy" conditions, or other adverse soil and seepage pressure conditions.

It should also be recognized that it may be impractical to provide absolutely safe channel slope designs for every foot of many channel sections. Surface and subsurface investigations, sampling and testing of soils at channel sites may not be as intensive as at dam sites. Stability analyses and slope design will generally have to be based on dominant conditions with adequate provision for maintenance of trouble spots that may show up during or after construction or that may not be large enough to warrant variation in the design of the overall project. However, in some situations where seepage conditions may be limited in extent and in critical areas, the design based on dominant conditions may be modified by drainage appurtenances or by a change in slope inclination.

The banks of existing channels in similar soils and under similar conditions should be studied. Past experience with channel banks under similar conditions should be reviewed.

Table 6-5 - Bedload Sediment Transport - 3 Stream Sections

Hydrograph Data (From Fig. 1)				28' Bottom Width-Natural Channel Slope-0.0195 ft/ft 1/2:1 Side Slopes		8' Rectangular Lined Channel Slope-0.014 ft/ft		14' Rectangular Lined Channel Slope-0.006 ft/ft	
Range In Discharge cfs	Mid-Point of Range cfs	Elapsed Time Hrs.	Time % of 24 Hrs.	Sediment Transport Per ft. width tons/day	Bedload Sediment Transport Col. 4 x Col. 5 x width tons	Sediment Transport Per ft. width tons/day	Bedload Sediment Transport Col. 4 x Col. 7 x width tons	Sediment Transport Per ft. width tons/day	Bedload Sediment Transport Col. 4 x Col. 9 x width tons
1	2	3	4	5	6	7	8	9	10
0-200	100	1	4.2	54	64	650	218	61	56
200-400	300	0.7	2.9	200	218	2,550	591	238	97
400-555	460	0.4	1.67	332	155	4,200	560	410	96
555-450	480	0.4	1.67	360	168	4,600	614	432	101
450-250	360	1.0	4.2	250	295	3,100	1,040	303	178
250-160	205	0.5	2.08	120	70	1,550	258	145	42
160- 25	72	1	4.2	36	42	460	154	40	24
25- 0	12	0.5	2.08	6	--	60	10	---	--
					1,012		3,435		574

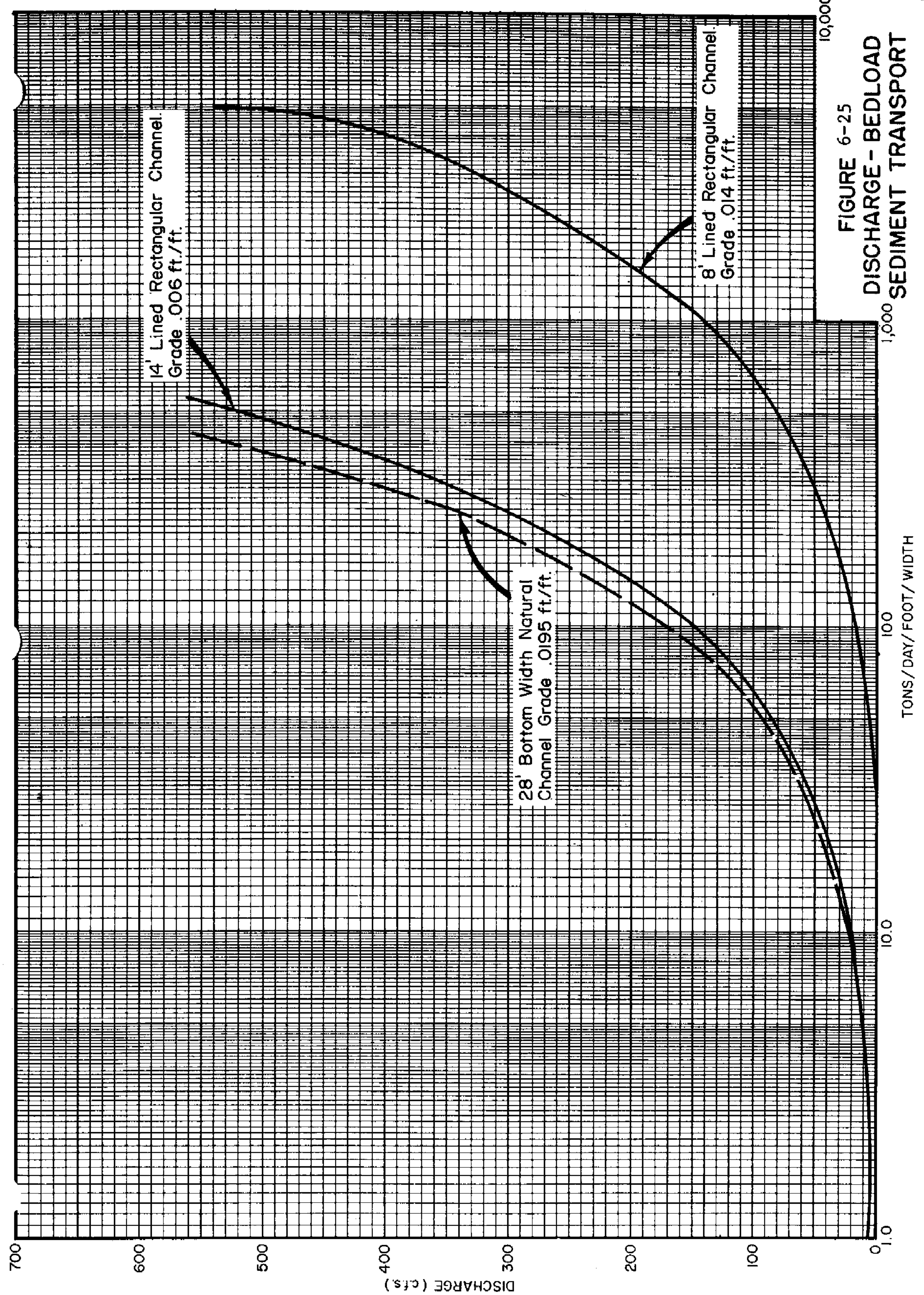


FIGURE 6-25
DISCHARGE-BEDLOAD SEDIMENT TRANSPORT

The design of most channel banks from the standpoint of strength will probably depend largely upon local experience and past performance; detailed analyses generally will be limited:

1. to those sections where critical soil or stress conditions are anticipated,

2. to new areas in which experience is lacking,

3. to high hazard areas where failure would cause severe damage.

Too often instability of channels is blamed on erosional activity or on bank sloughing. Actually, many channel bank failures involve a combination of erosion and shear failure, such as:

1. degradation of the channel bottom,

2. undercutting of a bank because of channel obstructions, improper curvature, or other factors that direct channel currents toward the bank,

3. loss of toe support for a slope from internal erosion (piping).

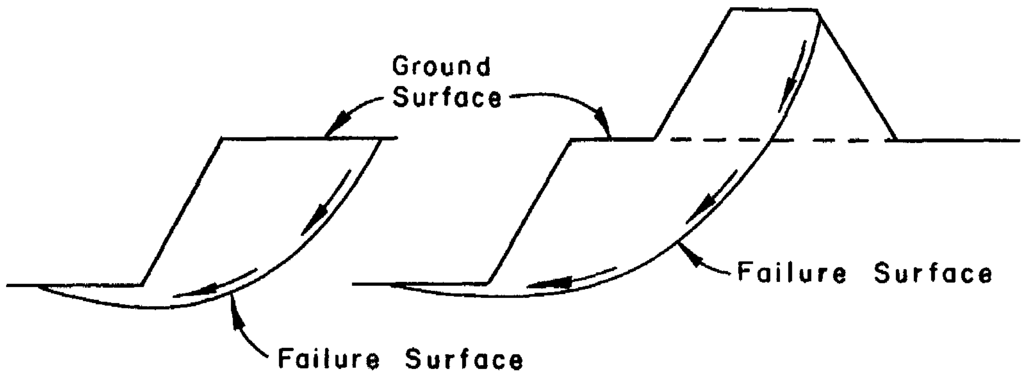
In these cases, the resistance to sliding is reduced, and the possibility of slope failure is increased, even though the original slope was stable before erosion occurred. These factors should be carefully considered in evaluating past performance of channels.

The object of a stability analysis is to determine the factor of safety for the most critical combination of stresses and boundary conditions anticipated. A good estimate of the location of the critical surface can usually be made by considering that the failure surface will tend to follow the path of least resistance, e.g., through or along material with the lowest shear strength.

Types of Slides and Methods of Analysis

No one method of slope stability analysis is applicable to all conditions. The type of potential slope failure and the location of the critical zone or plane of weakness generally dictate the method of analysis to be used.

Rotational slides. - - Rotational slides are those in which the sliding soil mass moves on a circular arc failure surface through any section of the slope or the channel bottom.



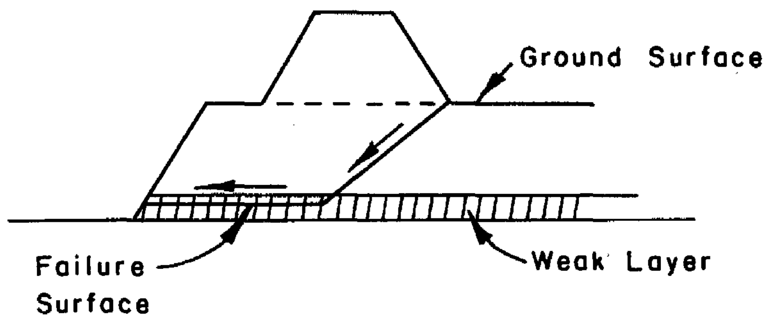
This type of failure generally occurs in plastic soils.

A Swedish slice method[34] is used to analyze rotational slides on circular failure arcs through levees, natural banks, or combinations thereof. This method is most applicable to soils that exhibit cohesion; it may be used in the no seepage, steady seepage and rapid drawdown conditions.

Although Taylor's[34] charts of stability numbers do not consider seepage, these charts may be used for rough determinations and preliminary solutions in homogeneous cohesive soils, provided a conservative factor of safety is used. If a channel is stable from an erosional standpoint and deep drying cracks are not likely to occur, bank stability in homogeneous cohesive soils is generally not a problem for channel depths less than about eight feet.

Janbu[35] has developed an analytical method for cohesive materials for cases of $\phi = 0$ and $\phi \neq 0$. Charts and graphs are presented for wet slopes in which submergence and drawdown are considered and for dry slopes. Effects of surcharge and tension cracks are also included. This method is limited to homogeneous levees, with single layer foundations or base materials, and no seepage.

Translatory slides. - - Translatory slides are those in which the soil mass moves on a zone of weakness that can be identified as the base for sliding.



This type generally occurs as a slide in a weak clay seam (stratum) or where uplift pressure is excessive.

1. The sliding wedge method[36] is applicable to translatory slides along weak planes at or above the bottom of the channel and roughly parallel to the ground surface. This method is used in connection with either the no seepage or the steady seepage conditions; it is difficult to evaluate the pore water effects in this method.

2. The infinite slope method of analysis is applicable to slopes of non-cohesive materials that are subject to either steady seepage or rapid drawdown conditions. This method of analysis assumes the soil mass slides parallel to the slope.

Boundary Conditions and Parameters Affecting Slope Stability

Effects of Water

The stability of channel banks is affected by the amount of water in the soil mass, the pressure head on the water, and the directional movement of water in the pores of the soil. The weight of the soil varies with moisture content which in turn affects the forces acting on the soil mass. The effect of moisture in terms of pore pressure alters the resistance of soil to sliding failure, i.e., seepage pressure will lower strength whereas surface tension in moist soil will increase the strength in relation to the saturated strength.

Steady seepage. - - When gravitational water moves through saturated soils, seepage forces are set up by the frictional drag exerted on the soil particles. These forces are functions of the head losses or hydraulic gradient through the soil mass. When flow moves from the bank into the channel, the resultant seepage forces decrease the stability of the bank.

The maximum anticipated elevation to which ground water may develop or the maximum pressure that may develop in shallow aquifers should be considered. The water level and pressure conditions at the time of investigation may not be the most critical condition. For example, downstream from a storage dam the ground water level in the valley may rise considerably after the dam is constructed. When water table conditions are involved in channel bank stability studies, water movement should generally be considered in a horizontal direction. Banks consisting of fine sands and non-cohesive silts are especially prone to slough under high water table conditions; stability of banks in these materials may not be achieved until the ground water has been lowered.

Under conditions of permanent low ground water where no seepage flow is assumed out of the bank, seepage forces can be neglected in the slope stability analysis. However, prolonged heavy rains can saturate a portion of the soil profile, especially if the profile is stratified, and bring water forces into consideration.

Drawdown. - - Drawdown is the lowering of the water level against a channel bank. When the water stands for some time against an earth slope, such as an irrigation canal, the soil becomes saturated. Rapid drawdown presupposes a sufficiently quick withdrawal of the water in the channel so that the soil in the banks remains saturated. Outflow from the banks is considered to move horizontally.

Shear Strength

The results of drained shear tests produce the best strength parameters for stability analysis of channel banks when all seepage forces are considered. Shear strengths from saturated unconfined compression tests or vane shear tests may be used for highly plastic soils. Results from consolidated, undrained shear tests may be used in lieu of results from drained shear tests when the former are considered adequate and representative or when pore water pressures are measured.

Unloading by excavation and the subsequent weathering of some bank materials may lead to swelling, cracking, decrease in density, and loss of shear strength. Under these conditions, the shear strength obtained from tests on unweathered samples must be adjusted downward on the basis of knowledge of the material, past experience, and judgement.[37]

Unless shear tests have been made on materials in spoil banks or levees, the shear strength of these materials should be ignored in the resisting forces. The weight of such materials must be considered in the driving forces, however.

Seismic Forces

The effect of earthquake shocks can be ignored in the stability analyses of channel banks in a large portion of the United States. However, in some areas (the Western States in particular) seismic effects should be evaluated as a design factor, when a slide would result in costly property damage or loss of life.

The designer should review the following references for information on earthquake history and seismic effects on dams:

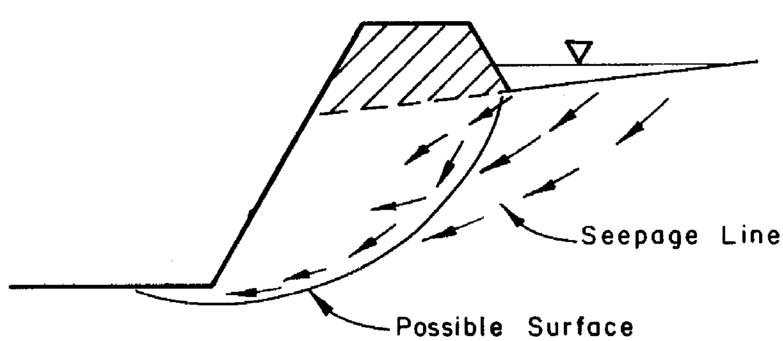
> "Earthquake History of the United States," U. S. Department of Commerce, Coast and Geodetic Survey Bulletin No. 41-1.
>
> "Seismic Stability of Earth Dams" by E. E. Esmiol, U.S.B.R. Technical Memorandum No. 641.

It is suggested that seismic loadings be obtained from Figure 17 of Technical Memorandum No. 641. These loads are assumed to act horizontally in the direction of instability and should be applied to the worst condition other than rapid drawdown.

In lieu of values from Figure 17, earthquake effects may be included by the addition of a horizontally directed inertial force of 0.1 g, i.e., the stress increase is 0.1 of the weight of material above the slip surface.

Surcharge

Surcharge loads, such as levees, spoil banks and roadways near the top of channel banks should be avoided or minimized when possible, especially those conditions shown in the figure below. In this situation, the driving forces are increased by the weight of the excavated material placed at the top of the bank. In addition, free runoff of surface water is prevented. The seeping water from the land side of the spoil bank weakens the soil in the zone of possible failure and increases its unit weight. The resisting forces are decreased, and the driving forces are increased.



When levees or spoil banks are located away from the edge of the channel bank so as to leave a berm at the ground surface, the forces tending to cause shear failure or sloughing of the channel bank are considerably reduced. NEH, Section 16[2], pages 6-18 and 6-19, contains a discussion on natural ground berms and spoil banks.

If surcharge loads will exist, they should be considered in the stability analysis of channel banks. The largest anticipated value of the unit weight of soil in levees and spoil banks should be used in the analyses. Unit weights will vary with soil types, moisture contents, and methods of placement. For example, the unit weight of materials placed by dragline may vary considerably from the unit weight of material placed by hauling equipment.

The line load, plus the appropriate roadbed surcharge load, should be included in the stability analysis when roads will be located adjacent to banks, on berms or on levees of channel projects.

Tension Cracks

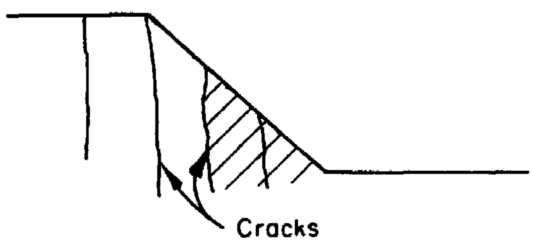
Experience has shown that the upper portion of most cohesive slopes is initially in a state of tension. The depth to which tension extends can be roughly estimated by the following equation:

$$H_c = \frac{2\ c_m}{\gamma_m} \tan(45° + \frac{\phi m}{2})$$

where terms are as defined in glossary.

In a stability analysis, the depth of tension cracks should not be extended below the water table or over one-half the height of the slope. For a vertical bank, the location of greatest tensile stress is back from the edge of the cut a distance equal to approximately one-half of the height. The cohesion portion of the shear strength should not be used in the zone of cracking. The hydrostatic pressure created by water in the cracks should be added to the driving forces.

Cracks caused by excessive shrinkage may exist in some soils to greater depths than the tension zone previously discussed. The soils generally most susceptible to shrinkage cracking are clays having liquid limits greater than 40 and plasticity indexes greater than 20.



In clay soils where the water table is low, shrinkage cracks may develop in channel banks as shown in the above sketch; regardless of the slope of the bank. Blocks of soil, as indicated by the cross-hatched lines, are further weakened by water in the channel and eventually are moved from the bank. Under these conditions, channel banks tend to become vertical. A similar situation occurs in dry soils having a columnar structure, such as loess.

None of the methods of stability analysis presented in this section are directly applicable to a solution involving the shrinkage condition.

Factors of Safety Against Sliding

The end result of all methods of slope stability analyses is a comparison of the forces that cause sliding with those that resist sliding. The ratio of the resisting forces to the driving forces is the factor of safety against sliding.

The minimum acceptable factor of safety is dependent:

1. on the method of analysis used,

2. on whether all loads and forces on the banks have been considered and included in the analysis,

3. on strength parameters that may have to be correlated or estimated to a considerable extent because of limited intensity of investigation and testing.

Piping

Piping is the movement of soil particles by percolating water and the subsequent development of internal channels or pipes. The formation of pipes in the periphery of a channel reduces support for the toe of the slope; this loss of support may eventually create an unstable bank.

When hydrostatic pressure exists in a sub-stratum at a planned project site, this pressure may become excessive when overburden materials are removed, with the result that heaving and piping may occur in the bottom and/or banks of channel. After excavation of the channel, the effective weight of the soil overlying the stratum under hydrostatic pressure must be greater than the uplift pressure if the channel is to be stable. In order to make an analysis, the hydrostatic pressure must be determined by piezometers or other means.

Water impounded in a reservoir may increase the uplift pressure on a channel downstream from a dam. In those cases where a less pervious blanket overlies a more pervious stratum, the uplift analysis may be made by blanket equations.[38]

In the case where an aquifer lies above the bottom of a channel, it may be necessary to construct a flow net to determine the exit gradient for use in a piping analysis.

The minimum acceptable factor of safety against heaving and piping is generally 1.5.

Stabilizing Measures

General

When the preliminary design for an earth channel indicates that the allowable tractive force and velocity will be exceeded, consideration should be given to vegetation or structural stabilization.

Stabilizing measures can be classified broadly into three groups - - bank protection, channel linings, and grade control structures.

Bank protection and channel linings protect the channel surfaces from erosion caused by movement of water and transported materials and from shallow surface sliding.

Grade control structures may be used to reduce the channel bottom grade with a resulting reduction in velocity and scour, to control overfalls at the head end of channels, and to control the discharge from tributary channels.

The selection of a particular measure or combination of measures should be based on sound engineering and agronomic principles for each particular situation since channel stabilizing problems can vary considerably from one location to another.

Except in narrowed channels, protective elements should approximate natural roughness. Revetments should be as coarse in texture as natural banks. Retards, baffles and jetties should simulate the effect of trees and boulders along natural banks and in overflow channels.

Bank Protection

Under certain conditions channel stability may be obtained by providing protection to the banks only. Examples are at sharp changes in alignment and at bridges, culverts, or grade control structures where the bottom is stable.

Vegetation[21/] [39/] [40/]

Vegetation may be considered for sites suitable to good vegetative growth. It can be used alone or in conjunction with structural measures to provide a more effective and permanent type of protection.

It may be necessary to use temporary materials to protect the seedlings or plants against erosion from wind and runoff during the period of establishment.

Permissible velocities for vegetative cover are given in Table 3 of SCS-TP-61[21/]. This table indicates a range of velocities from 2.5 f.p.s. for easily erodible soil, to 8.0 f.p.s. for erosion-resistant soil. Velocities exceeding 5.0 f.p.s. should not be used except where good cover and proper maintenance are assured.

Conditioned Earth

Conditioned earth may be used to increase the stability of channels with stable bottoms by providing denser, more erosion-resistant soil in the channel banks. Earth banks may be "conditioned" by the following methods: (See page 29 of [39/])

1. Compacting the existing soil in the shaped channel to a greater density.

2. Over-excavating to a larger cross section than necessary, and placing a compacted less permeable soil as a lining. Both methods 1 and 2 are not easily adaptable to slopes steeper than 3 to 1.

3. By adding chemicals to the soil.

Revetments[41/] [42/] [43/]

Revetments of various types may be used to stabilize channel banks.

Retards and permeable jetties. - Retards and permeable jetties are extensive or multiple-unit structures composed of open forms like piling, fencing, and unit frames. However, their function and alignment are different.

Retards are placed parallel to erodible banks of channels on stable gradients where the prime purpose is to lessen the tangental or impinging stream velocities sufficiently to prevent erosion of the bank and to induce deposition. As a remedial measure, the prime purpose may be deposition near the bank in deep channels or restoration of an eroded bank by accretion.

Retards may be used alone (see Fig. 189 of [43/]) if the bank will be protected by deposition behind the retard, or by establishment of vegetation, otherwise they should be used in combination with an armor protection. (See Fig. 190 of [43/]) Retards may permit use of a lighter type of armor or they may be used as toe protection of armor revetments when a good foundation for the revetment is impractical because of high water or extreme depths of poor soil materials.

On tangent reaches where the channel is narrow, retards may, by slowing the velocity on one side, affect an increase in velocity on the other. In wide reaches of a meandering stream retards may reduce an impinging attack as well as have beneficial effect on the opposite bank by slowing a rebounding high velocity wave.

Permeable jetties are placed at an angle with the channel bank and are generally used in meandering streams to direct the current away from the bank. (See Fig. 191 of [43]) They encourage deposition of bed material and growth of vegetation, but where retards build a narrow strip in front of the bank, permeable jetties cover a wider area roughly limited by the envelope of the outer ends.

Timber piling. - - Timber piling retards and jetties may be of single, double or triple rows of piles with the outside or upstream row faced with wire mesh or woven wire fencing material which adds to the retarding effect by trapping light brush or debris. This type of retard is particularly adapted to larger channels where the piles will remain in the water, removed from the fire hazard of brushy banks. The number of pile rows and amount of wire may be varied to control the deposition of material. In leveed channels, it is often desirable to discourage accretion in order not to constrict the channel but still provide sufficient retarding effect to prevent loss of bank protection such as vegetation or small rock riprap. When used as jetties, the purpose is to encourage deposition of material and protect vegetation. Assuming negligible fire hazard, the wood may be treated with preservative to provide a long life (Fig. 195 of [43]).

Fence types. - - For smaller channels or areas of less frequent flood flow attack, such as overflow areas, single and double rows of various types of fencing may be used. (Figs. 202-205 of [43]) All metal types, such as pipe-and-wire or rail-and-wire, are more suitable when conditions are conducive to the growth of brush that presents a serious fire hazard to wooden posts. Details of typical designs of pipe-and-wire retards are found in Figs. 206 and 207 of [43].

The principal difference between fence retards and ordinary woven wire fences is the posts of retards must be driven sufficiently deep to avoid loss by scour.

When it is necessary to reduce the permeability as an aid in directing the stream, as is frequently required at earth fills behind bridge abutments, self-adjusting wire baskets[43] may be used and filled with alternate layers of rock and brush.

Permeability can be varied to meet the requirements of the location. For single fences, the factor most readily varied is the pattern of the wire mesh. For multiple fences, the mesh pattern can be varied or the space between fences can be filled to any desired height.

Making optimum use of local materials, this fill may be brush ballasted by rock, or rock alone.

Jacks and tetrahedrons. - These devices are skeletal frames adaptable to permeable retards and jetties by tying a number of similar units together in the desired alignment.

They serve best in meandering channels that carry considerable bedload during flood stages. Impedance of the stream along the string of units causes deposition of bed material, especially at the crest of flow and during falling stages. Beds of such streams often scour on the rising stage, undercutting the units and causing their subsidence, and rotation when one leg or side is undercut more than the other. Deposition on the falling stage usually restores the former bed and partially or completely buries the units. However, in the lowered and rotated position, they may be completely effective during future flood flows.

Selection of jacks and tetrahedrons may be influenced by location in or near urban or recreational areas. Unless the units will be screened by natural vegetation, attention should be given to their appearance. Where units may become "attractive nuisances," details should avoid sharp points and edges or other features dangerous to children.

Rock riprap (not grouted). - This kind of protection consists of rock courses placed either directly upon the bank slopes or on gravel filters on bank slopes. (See Fig. 152 of 43/)

Where stones of sufficient size and quality are available, it may be the most economical type of revetment and has the following advantages:

a. It is flexible.

b. Local damage or loss is easily repaired by the addition of rock.

c. Appearance is natural, hence acceptable in recreational areas.

d. Vegetation may grow through the rocks adding structural value to the bank material and restoring natural roughness.

e. Additional thickness can be provided at the toe to offset possible scour when it is not feasible to found it upon a solid foundation.

f. Wave runup is less (as much as 70%) than with smooth linings.

g. It is salvable. The rock may be stockpiled and re-used if necessary.

h. Rock slope protection, more than any other type, adopts a non-uniform widely varying material to a structural purpose, with gravity alone holding the stone together.

Rock riprap should:

a. Assure stability of the protected bank as an integral part of the channel as a whole. For this major objective, the ideal condition for stability is a straight channel or a gently curved channel with its outer bank rougher and more erosion resistant than the inner bank.

b. Tie to stable natural bank, bridge abutments or other fixed improvements with transitions designed to ease differentials in alignment, grade, slope and roughness of banks.

c. Eliminate or ease local irregularities so as to streamline the protected bank.

<u>Rock and wire mattress (gabion revetment)</u>. - This type of bank protection consists of connected flat mats fabricated of wire mesh or woven wire fencing filled with rock and adequately anchored to the bank. (See Fig. 363 of [43])

As a revetment, its application has been limited to locations where the rock economically available is too small for ordinary rock riprap, or where grouted protection is unsuited because of fineness of stone or insecurity of bedding or foundation. Alternatives of wire strength and mat sizes make rock and wire mattresses adaptable to a wide range of exposure to hydraulic forces, but the lighter exposures are served more economically by reticulated revetment.

The most common use of rock and wire mattresses has been to provide flexible toe protection for other types of bank protection as shown in Figs. 179-181 of [43]. The mat will adjust itself by flexure and subsidence, and block the progress of erosion and scour that might threaten the toe of the bank. This type has not performed well on curves (Figs. 182, 325 of [43]), where settlement requires extending or shortening of the length of the mat. It is more adapted to tangent reaches when the mat has sufficient strength to hang suspended when deep or uneven scour occurs. Its life and that of the bank protection above depend on the durability and strength of the wire. Therefore, rock and wire mats should have a longer service life in drier climates and mature channels carrying mud and silt (but not gravel and stones that would abraid and shorten the life of the wire mesh).

Considering the high cost of the labor involved, the questionable service life of the wire, and the efficiency of modern methods of excavating for toe protection, use of this type of bank protection has declined.

Reticulated revetment. - Wire-mesh netting is useful in revetment work to confine rock that by itself would be too light to resist the erosive forces of the stream flow. It may be used as a cover for banks over which a layer of rocky material has been placed. The size of the mesh must be small enough to confine the majority of the stones. Although some small stones may wash through the netting, there will remain a top layer of larger stones which, in turn, will confine the small stones underneath.

The netting is placed over the rocky slope and pinned by means of short lengths of reinforcing bar hooked at the top. Brush may grow through the wire and provide additional anchorage. An application of this type is shown in Fig. 187 and typical design details are shown in Fig. 188 of [43]. If the channel bed material is gravel, the wire may serve as a flexible toe protection by extending it into the channel bed and weighting the toe end.

Sacked concrete riprap. - This method of protection consists of facing the banks with sacks filled with dry concrete mix. Much hand labor is required but it is simple to construct and adaptable to almost any contour. A photograph of this type of installation is shown in Fig. 169 and typical plans adapted to several slopes are reproduced in Figs. 170 and 171 of [43].

Sacked concrete is an expensive but commonly used type of revetment. Where both ledge rock and gravel are readily available, sacked concrete may cost four to five times as much as an equal quantity of rock. It is almost never used unless suitable stream gravel is available at the location and satisfactory rock is not.

Dry pack may be an excellent device for subaqueous placement, for initial foundation, or repair of undercuts. It is also adaptable for protection or repair of small areas.

In many locations, the smoothness of sacked concrete is very undesirable and its use may require surface roughening. Projecting dowel bars and honeycombed surface concrete have been used for this purpose.

Portland cement concrete articulated block. - This type of revetment consists of small precast concrete blocks held together to form a flexible mat. A typical installation is shown in Fig. 115 of [43].

In this type of installation, the blocks contain wire-mesh reinforcement with rebars extending out from each edge and bent into an eye at one edge and a hook at the opposite edge. As the block is placed, the open hooks are put through the eyes of the adjacent blocks and closed. It is easily placed and is desirable from an appearance standpoint. This type of fabrication becomes complicated for curved contours as the blocks must be cast in different sizes for each row. Use of this type has been most successful for toe protection on tangent sections.

Grouted rock riprap. - This type of revetment consists of rock riprap having voids filled with portland cement concrete grout to form a monolithic armor. A photograph of this type of installation is shown in Figs. 112, 159 and a typical plan in Fig. 160 of [43]. It has application in areas where rock of sufficient size for ordinary rock riprap is not economically available. It also will generally reduce the quantity of rock needed for a given job. Grouting not only protects the stones from the full force of high velocity water but integrates a greater mass to resist its pressure.

Grouting will usually more than double the cost per unit volume of stone, but the use of smaller stones in grouted rock slope protection than in an equivalent protection using ungrouted stones permits a lesser thickness of protection which may offset to some extent the cost of the grout.

As this type of protection is rigid without high strength, support by the banks must be maintained. Slopes steeper than the angle of repose of the bank material are risky.

Asphalt concrete paving. - This type of revetment consists of a facing of asphalt concrete usually reinforced by wire mesh. (See Fig. 175 of [43]) Such revetment is very susceptible to damage from hydrostatic pressure behind the pavement and should not be used unless relief from this condition can be provided at reasonable cost.

It has found most use in bank linings where drawdown is not rapid and water pressure acts to maintain close contact between the paving and the bank. It has been used without reinforcement as a lining for small drainage ditches where it is placed and compacted by hand. (See Fig. 176 of [43].)

Concrete paving. - This method of protection consists of paving the bank slopes with reinforced portland cement concrete. A photograph of this type of installation is shown in Fig. 162 and typical plans are shown in Figs. 163 and 164 of [43].

It is particularly adaptable to locations where the hydraulic efficiency of smooth surfaces is important. On a cubic yard basis, the cost is high but as the thickness is generally only 3 to 6 inches, the cost on a basis of area covered will usually be less than for sacked concrete slope protection. This is especially so when sufficiently large quantities are involved and alignment will permit the use of mass production equipment such as slip-form pavers.

Because of the rigidty of portland cement concrete slope paving, its foundation must be good and the bank slopes stable.

Bulkheads. - In bank protection, a bulkhead is constructed along a steep slope to retain the bank from sliding as well as to protect it against erosion. (Fig. 226 of [43].)

Walls. - - The commonest bulkhead in bridge practice is the wingwall (or endwall) serving as a transition from a rectangular constriction to a trapezoidal channel. The commonest forms (Fig. 227 of [43]) are:

1. Straight Endwall

   This type has no transitional value but protects approach against eddy erosion; it is suitable only for low velocity in poorly defined channel.

2. Straight Wingwall

   This type also has no transitional value but protects steep banks which support the approach embankment.

3. Oblique Wingwall

   This is a conventional transition; it is efficient and economical for well-defined channels and moderate velocity. Flare angle in degrees should be limited to $\frac{300}{V}$ for converging and $\frac{150}{V}$ for diverging flow, where V is the velocity in f.p.s. through the constricted section.

4. Tapered Wall

   Tapering the grade of the parapet of either the straight or oblique wingwall is common practice for streams of moderately low velocities. By matching the surcharge slope to the natural bank, the transition progressively exposes this slope to the low velocity boundary of the varied flow.

5. Warped Wall

   Tapering the slope of the wall from vertical at the abutment to a stable-bank slope at the end of the wall makes an excellent transition for moderate to high velocity.

6. Returned Wall

Building the standard cantilever wall on a curved alignment returned from the abutment is an economical solution for a combination of a vulnerable approach embankment projecting into a channel with durable banks.

Cribs. - - Timber and concrete cribs are used for bulkheads in locations where some flexibility is desirable or permissible (Figs. 229-233 of 43/). Using backfill for stability, cribs are economical in the use of structural materials. Their rough surfaces are advantageous in all natural locations where banks are exposed to high velocities.

Piling. - - Timber, concrete and steel piling are used for bulkheads depending on deep penetration of foundation materials for all or parts of their stability. Any of the three materials is adaptable to sheet piling or a sheathed system of post or column piles. (Fig. 234 of 43/.)

Channel Linings

Channel linings are used to protect the entire channel surface.

Vegetation is the most commonly used protection for channels with infrequent flow, relatively low velocities, and where a good stand can be established and maintained.

Ungrouted rock riprap may be used for channel lining where soils are not suitable to vegetative growth. Such lining is applicable to the inlet and outlet of channel structures for stabilization of bottom and banks.

Where it is necessary to conserve water by limiting or eliminating seepage, where high velocity flow occurs, or where channel operation at high hydraulic efficiency is required, durable, relatively impervious linings such as concrete or asphaltic concrete may be used. For relatively short reaches grouted rock riprap may be used. Such linings may be required where channel right-of-way is limited.

The usual shape of cross sections for vegetated cover, ungrouted rock, grouted rock, asphaltic concrete, or other non-structural sections is trapezoidal. For reinforced concrete, the cross section may be rectangular or trapezoidal. The type of protection selected will depend almost entirely on economics. The only sure way to select the most economical material is to prepare a preliminary design for each and compare annual costs.

As a guide to trial selection of the type of lining, the following approximate criteria are presented:

247-927 O - 78 - 15

1. Rectangular reinforced concrete channels will show the least annual cost when velocities are high, rights-of-way are expensive, and wall heights are 15 feet or less.

2. Trapezoidal R/C channels are most economical for the above conditions when right-of-way costs are more moderate and channel wall heights are quite great (usually over 15 feet).

3. Loose rock lining is efficient when velocities are not so great as to require extremely large rock and thick sections, and where rock and filter material are available from nearby sources.

4. Grouted rock lining is generally economical only for short reached of high velocity flow where extremely large rock would be required for loose rock lining.

5. Channel linings constructed of asphaltic concrete, pneumatically applied mortar, pre-cast R/C slabs are usually economical on an annual cost basis only in special situations of availability, short project life requirements, etc.

Grade Control Structures[43/] [44/] [45/] [46/] [47/]

Various types of structures may be used to reduce the gradient in channel reaches where the channel materials will not resist the erosive forces. They can be divided into two classes - - open top structures and closed conduit structures.

Open top structures, such as drop spillways and chutes, may be considered for use with any size channels.

Closed conduit structures, such as culverts, hooded inlets and drop inlets are generally used in relatively small channels.

In the design of grade control structures the site configuration, foundation, conditions, availability of construction materials, hydraulic and structural adequacy, and economic factors should be considered.

Generally, the design procedure for grade control requirements should include the following:

1. For the channel reach selected, determine the total fall between upstream and downstream limits.

2. For the design discharge, the selected channel dimensions and type of channel protection generally determine the maximum stable channel gradient.[20/] [21/]

3. Using the total fall, length of channel reach, and stable grade, determine the amount of fall to be controlled by structure(s).

4. Select the type and size of grade control structures needed, based on site configuration, foundation conditions, availability of construction materials, hydraulic and structural adequacy and economic factors.

Since channel dimensions and type of protection directly affect the stable grade and the amount of fall to be controlled by structures, alternate designs should be made to select the most practical and economical overall plan.

Open Top Structures

Straight drop spillway[4/] [46/]. - - This type of structure is efficient for the control of relatively low heads normally up to 10 feet. It is very stable for heads less than 10 feet and the likelihood of serious structural damage is more remote than for other types of structures. However, a stable grade below the structure is essential to stability.

A rectangular weir is less susceptible to clogging by debris than the openings of other structures of comparative discharge capacities. When properly constructed, maintenance costs are lower for straight drops than for other types of grade control structures for most embankment and foundation soil conditions. It is relatively easy to construct.

Limitations to the use of the drop spillway are:

a. It is more costly than some other types of structures where the required discharge capacity is less than 100 c.f.s.

b. When the total head or drop is greater than 10 feet, it becomes costly to stabilize this type of drop structure against sliding.

c. It is not a favorable structure where it is desired to use temporary spillway storage to obtain a large reduction in discharge.

Box inlet drop spillway[46/]. - - The box inlet drop spillway can be used for the same purposes as a straight drop spillway. One of its greatest uses is for grade and erosion control in open channels where the width of outlet is limited. It can also serve as a tile outlet at the head end of the channel.

It is particularly adapted to narrow channels where it is necessary to pass large flows of water. The long crest of the box inlet permits large flows to pass over it with relatively low heads, and the width of the spillway need be little, if any, greater than that of the exit channel. The box inlet drop spillway can be easily combined with a bridge to provide a road crossing. The high portion of the sidewalls can be used as abutments for the bridge.

The structural design of the box inlet drop spillway is more complex than for straight drop spillways.

Island-type spillway[46/]. - - The island-type spillway consists of a drop structure in the channel with earth emergency spillways for carrying storm flow around the structure. Either the straight drop spillway or the box inlet drop spillway can be used. When the weir length of the structure is greater than the bottom width of the channel, the box inlet drop spillway should be considered. This type of spillway is adaptable for use at the head end of channels to control the overfall. It is particularly adapted to site conditions where the design runoff volume is greater than the capacity of the outlet channel into which the structure empties. The use of this type of grade control structure is limited to areas where there is sufficient nearly level land on either side of the channel for use as earth spillways. Topography of the ground must be such that the path of overflow around the structure will return to the channel locations a short distance below the structure without causing damage to the land or channel banks.

The island-type spillway is proportioned so that the channel will be full before the overflow around the dam enters the channel, thereby eliminating the possibilities of bank erosion from flow over the bank. To accomplish this, the crest of the weir must be set below the bottom elevation of the earth spillway, a distance sufficient to provide a weir notch capacity between these two points equal to the bank full capacity of the channel at the place where the flow from the auxiliary spillway will enter the channel. Larger flows will then pass around the earth embankment of the drop spillway forming an island composed of the drop spillway and the headwall extension levees. The waterway above the structure must have the same capacity as the channel below the dam at the point of overflow. The island spillway should be so proportioned that earth spillways will begin to flow as soon as the channel capacity flow has been reached. In order to force overflow water away from the dam and protect the fill from washing out around the dam, levees extending each way from the dam must be provided.

The island-type structure permits the use of a spillway having a capacity less than would be required to handle the total runoff peak discharge. It requires the construction of auxiliary spillways in areas that may be cropland where maintenance of the correct grade and elevation is difficult.

Concrete chute spillway[44/] [46/]. - - The concrete chute is particularly adapted to high overfalls where a full flow structure is required and where site conditions do not permit the use of a detention-type structure.

Chutes may be more economical than drop inlet structures of the same capacity and drop when larger capacities are required.

Closed Conduit Structures

Hooded inlet spillway[46/]. - - The hooded inlet spillway is best adapted for use at sites where the pipe can be installed in the original ground. Construction is complicated when the pipe is placed in the embankment.

The hooded inlet spillway will flow completely full for conduit slopes up to 36 percent (the limit of present tests) if the length of the hood is properly selected and the head on the inlet is sufficient. As compared with the drop inlet, it has the advantage that no riser is required and there is less fill over the pipe. It is simple to fabricate and install and is comparatively low in cost.

Drop inlet spillway[46/]. - - The drop inlet is an efficient structure in the control of relatively high heads. It is well adapted to sites providing an appreciable amount of temporary storage above the inlet. It may also be used in connection with relatively low heads, as in the case of a drop inlet on a road culvert.

For high heads, drop inlets require less material than a drop spillway under similar circumstances. Where an appreciable amount of temporary storage is available, the capacity of the structure can be materially reduced. Besides affecting a reduction in cost, this reduction of discharge results in a lower peak channel flow below, and can be a favorable factor in channel grade stabilization and flood control.

Drop inlets are subject to plugging by debris. They are limited to locations where satisfactory earth embankments and emergency spillways can be constructed.

Culvert drop box[46/]. - - Drop boxes are used to control gradients above culverts in either natural or constructed channels and, in addition, they may serve as an outlet structure for tile lines in drainage systems. Cattle ramps can be incorporated into the design of the box when the culvert is used as a cattle pass. The drop box is very effective for roadway erosion control.

The drop box is one of the most economical structures for controlling overfalls because the existing culvert and roadway embankment replaces the outlet portion of the typical drop spillway. It has the advantage of the box inlet drop spillway in that weir length can be fitted to a narrow waterway.

Other Structures

General

A comprehensive channel design frequently requires the incorporation of one or more of the following structures and/or practices:

1. channel crossings;

2. channel junction structures;

3. side inlet structures;

4. water level control structures.

Channel Crossings

Channel crossings are required where private or public roadways pass over the channel. Structures used for this purpose are stream fords, culverts, and bridges.

Stream fords[46/]. - - Stream fords are installed on the channel surface. They provide the most economical type of crossing. They can be constructed of reinforced concrete, compacted rock, or broken concrete.

Stream fords are best suited for use in the upper ends of channels. They should not be installed where deep flows of long duration will prevent normal use.

Culverts. - - Culverts of concrete or metal pipes also provide an economical crossing when used at locations where the flow is relatively small, and where serious resistance to the flow of water is not a limiting factor in overall channel design. For hydraulic design, see page 6-29 of [2/].

Bridges. - - Bridges of concrete or timber should be used when necessary on most open channels that are designed to capacity on low gradients. Since they do not offer serious resistance to the flow of water, they are preferred over culverts, especially for high flows. For hydraulic design, see page 6-32 of [2/].

Channel Junction Structures

Where two main channels join, wave formation can be minimized if the two flows at the junction are as nearly parallel as possible. The design criteria for structures at a junction of 2 trapezoidal or 2 rectangular channels is shown on Fig. 5-1.

Side Inlet Structures

Provisions should always be made for lowering surface water from adjoining fields to the main channel without serious erosion. Pipe spillways, drop spillways, and chutes are the more common types of structures used for side inlets.

Side inlet structures should empty into areas recessed in the banks of the main channel. Construction in this manner will minimize damage by the movement of floodwater, debris, or ice, and also will cause less retardance of flow in the main channel.

Pipe spillways[46/]. - - Pipe spillways can be used advantageously to convey water from bank of levees and continuous spoil banks into a channel. The hooded inlet is most efficient where discharge capacity is a problem. The flared inlet is less efficient but facilitates the passing of debris such as corn stalks and grasses. The pipe drop inlet is efficient and can be used as a tile outlet. When the required pipe size exceeds 48-inches in diameter, an open top structure should be considered for economy.

Drop spillways[46/]. - - Drop spillways are generally used where the volume of water to be handled is large. They can be used as a tile outlet structure. The drop spillway fits conditions where there is no spoil bank and functions well at the head end of a channel.

Reinforced concrete chutes[46/]. - - Concrete chutes function well where the volume of water to be handled is large and the overfall is such that a drop spillway will not be economical.

Vegetated chute[46/]. - - This type of chute should be limited to small watersheds and sites where good, dense sod can be developed and maintained. The water course below the chute must be stable. When the channel below the chute is narrow or conditions at the lower end of the chute may not be favorable to establish and maintain vegetation because of poor soil or rocky or wet conditions or siltation from adjacent channels or streams, a toewall should be used. The toewall will raise the end of the sod chute above these unfavorable conditions and permit the maintenance of a good vegetation. The toewall is a small drop spillway with a headwall generally 1 to 2 feet in height.

A vegetated chute is economical since material and construction costs are generally low. Use is limited to sites where the velocity of flow in the chute is low enough to maintain the vegetative cover. This generally limits the use of vegetated chutes to small water courses with low overfalls where there is no long, sustained flow.

Riprap chute[46]. - - A rock riprap chute provides a more stable outlet than a vegetated chute. The use of native rock may make it less expensive than a pipe or concrete structure of comparable size. It is a permanent type facility requiring less maintenance than a vegetated chute.

Rock riprap lined chutes are limited to areas where suitable durable cobbles or rock are available for construction. It requires careful adherence to the basic details of design in their construction to obtain satisfactory performance and stability.

Gabion chute[46]. - - The gabion chute is similar to the riprap chute except that the rock is placed in wire baskets. It is particularly adaptable to unstable foundation conditions because of its ability to adjust and retain its general section with displacement or compression of the foundation. The opportunity to fill it with native rock and cobbles makes its cost favorable in comparison with reinforced concrete. Generally, by the time the long-lasting wire baskets deteriorate, the structure will be so well established and bound together that it will remain indefinitely without the need for added protection.

Water Level Control Structures[46]

Water level control structures are used to regulate and maintain water in channels for water table control or for flooding land surfaces. The control is accomplished by use of gates or stop logs that can be fitted into several types of structures. The most common types used are drop spillways, box inlets or culverts, and open flumes.

Design Features Related to Maintenance

Channels must be properly maintained to function as designed. Maintenance can be made easier and more effective if certain features are incorporated in the design.[2]

Added Depth or Capacity for Deposition

Allowance should be made in the design for initial sloughing and sedimentation. Quite often during the first year after construction, the channel bottom will be raised from sloughings left by construction equipment. Soil and seepage conditions affect bank sloughing and silting. The sedimentation problem must be considered in the design so that depth and capacity will be provided over a period of years in line with economy.

Relationship of Side Slopes to Maintenance Methods

The slope of channel banks may be dependent on the type of maintenance as well as on soil conditions; for example, 3:1 slopes or flatter are usually needed for banks to be mowed.[2]

Berms

Berms may be used to facilitate maintenance by:

1. Preventing material from washing or rolling into the channel.

2. Providing work areas and facilitating spreading of spoil banks.

3. Providing access roadways.

Berm design may follow the general practice of the locality where the channel is to be constructed, provided proper loading and soil conditions are used. Guidance to minimum berm widths is given on NEH, Section 16[2], page 6-19, and National Standard and Specification Guide for Dikes and Levees.

Maintenance Roadways

Roadways should be provided for access to the channel with maintenance equipment and for inspection. They can be located on berms, spoil banks, or levees. On channels in excess of 20' top width, roadways may be required on both sides of the channel. The roadway should be wide enough to handle all maintenance equipment and should slope away from the channel.

Spoil

It is good practice to spread spoil banks to the extent that they can be maintained properly and can be used in the same manner as the adjoining area. The degree to which the spoil is spread depends upon the local conditions.[29]

Entrance of Side Surface Water to Channel

Side surface water should not be allowed to spill over the channel bank without protection. Interception ditches should be provided to control local drainage on the land side of the berms or spoil banks throughout the length of the project. These ditches should be graded toward collection points to drain into the channel through lined chutes, pipe drops and culverts, or over drop spillways.

Seeding

The berms and spoil should be seeded. Quite often the channel side slopes are also seeded. The extent to which seeding is done depends upon the location of the channel and local desires. Side slope seeding is accepted as good practice, particularly when flat side slopes are used so that both seeding and maintenance can be done economically.

Pilot Channels

Occasionally pilot channels are used to facilitate construction of a channel system as designed. The principal function of a pilot channel is to lower the water table sufficiently to permit deeper excavations to be made with greater safety and economy. This is accomplished by excavating the pilot channel as deep below the water table as practical without causing excessive sloughing of the banks. Construction is then deferred until the water table is lowered and the banks become more stable.

GLOSSARY OF SYMBOLS

A - alignment factor to adjust the basic velocity because of the effects of curvature of the channel.

A - area of flow. (ft^2)

b - bottom width of a channel (feet).

b_T - water surface width (feet).

B - bank slope factor to adjust the basic velocity because of the effects of different bank slopes.

C - sediment concentration in parts per million by weight.

C_1, C_2, C_3, C_4, C_5 - coefficients used to determine channel proportions and slope when using the modified regime equations.

C_e - Density factor to adjust the basic velocity because of variations in the density of soil materials in the channel boundary.

c_m - cohesion intercept at natural moisture (psf).

d - depth of flow (feet).

d_c - critical depth of flow (feet).

d_m - mean depth of flow (feet).

D - depth factor to adjust basic velocity because of the effects of the depth of flow.

D_s - the particle diameter of which s% of the sample is smaller.

F - frequency factor to adjust the basic velocity because of the effect of infrequent flood flows.

F - Froude number = $\frac{V}{\sqrt{gd_m}}$

g - acceleration due to gravity (fps^2).

G - specific gravity.

H_c - depth of tension crack (feet).

k_s - characteristic length of roughness element, for granular material.

$k_s = D_{65}$ size in feet.

K - coefficient modifying tractive force for gravitational forces on coarse, noncohesive materials on channel sides.

n - Manning's coefficient.

n_t - Manning's coefficient for roughness of soil grains.

P - wetted perimeter.

PI - Plasticity index.

q_u - unconfined compressive strength.

Q - discharge (cfs).

Q_s - sediment transport rate (tons/day).

R - hydraulic radius - feet

R_c - radius of curvature of central section of compound curve.

R_t - hydraulic radius associated with grain roughness of the soil.

s_o - slope of channel bottom.

s_c - critical slope.

s_e - energy gradient

s_t - rate of friction head loss because of tractive stress acting on bed and side materials.

V - average velocity (fps).

V_a - allowable velocity (fps).

V_b - basic velocity (fps).

V_c - critical velocity (fps).

W - average width of flow - ft.

W_T - top width of flow - ft.

x - factor describing effect of ratio $\frac{k_s}{\delta}$ on flow resistance.

z - cotangent of side slope angle.

T - factor to correct allowable tractive force for materials with D_{75} > 0.25" for unit weights different than 160 pcf.

γ - unit weight of water (pcf).

γ_d - dry unit weight (pcf).

γ_m - moist unit weight (pcf).

γ_s - unit weight of particles larger than 0.25" (pcf).

γ_w - unit weight of water (62.4 pcf).

δ - thickness of laminar sublayer $= \dfrac{11.6\nu}{\sqrt{gR_t s_e}}$

ϕ - angle of shearing resistance.

ϕ_m - angle of shearing resistance at natural moisture content.

ϕ_r - angle of repose of coarse noncohesive materials.

ν - kinematic viscosity of water (ft^2/sec).

ρ - water density ($lb\text{-}sec^2/ft^4$).

τ - reference tractive stress (psf).

τ_∞ - tractive stress in an infinitely wide channel (psf).

τ_b - maximum tractive stress on the channel bed (psf).

τ_s - maximum tractive stress on the channel sides (psf).

τ_{bc} - maximum tractive stress on the bed in a curved reach (psf).

τ_{sc} - maximum tractive stress on the sides in a curved reach (psf).

τ_{Lb} - allowable tractive stress along the bed. (psf)

τ_{Ls} - allowable tractive stress along the sides (psf).

REFERENCES

1. Marmer, H. A. - "Tidal Datum Planes", U. S. Coast and Geodetic Survey Special Publication No. 135; Gov. Printing Office, Rev. 1951.

2. Soil Conservation Service - National Engineering Handbook, Section 16, Drainage.

3. Soil Conservation Service - National Engineering Handbook, Section 6, Structural Design.

4. Soil Conservation Service - National Engineering Handbook, Section 11, Drop Spillways.

5. Soil Conservation Service - National Engineering Handbook, Section 8, Engineering Geology.

6. Soil Conservation Service - National Engineering Handbook, Section 9, Foundations and Earth Structures.

7. ASAE - Spec. Pub. SP-SW-0262, Measuring Saturated Hydraulic Conductivity of Soils.

8. Bureau of Reclamation - Earth Manual, First Edition, Denver, Colo., 1960.

9. Flaxman, E. M. - Channel Stability in Undisturbed Cohesive Soils; ASCE Hyd. Paper No. 3462, March 1963.

10. Middleton, H. E. - Properties of Soils Which Influence Soil Erosion; USDA Technical Bulletin No. 178, March 1930.

11. Smerdon, E. T., and Beasley, R. P. - Critical Tractive Forces in Cohesive Soils; Missouri Agricultural Experiment Station, Journal Series No. 2082, 1959.

12. United Nations Economic Commission for Asia and the Far East - River Training and Bank Protection; Flood Control Series No. 4, Bangkok, 1953.

13. Simons, D. B., Richardson, E. V., and Haushild, W. L. - Some Effects of Fine Sediment on Flow Phenomena; USGS Water Supply Paper 1498-G, 1962.

14. Leliavsky, Serge - An Intorduction to Fluvial Hydraulics; 257 pp., Constable and Co., Lt., London, 1955.

REFERENCES (Continued)

15. Lane, E. W., and Borland, W. M. - River and Bed Scour During Floods; Trans. ASCE, Vol. 119, 1954; pp. 1954; pp. 1067.

16. Kuenen, Ph. H. - "Sand"; Scientific American, April 1960; pp. 95-110.

17. Soil Conservation Service - National Engineering Handbook, Section 4, Hydrology, Part 1, Watershed Planning.

18. Soil Conservation Service - National Engineering Handbook, Section 15, Irrigation.

19. U. S. Weather Bureau - Technical Paper No. 40, Rainfall Frequency Atlas of the United States.

20. Soil Conservation Service - National Engineering Handbook, Section 5, Hydraulics.

21. SCS-TP-61 - Handbook of Channel Design for Soil and Water Conservation; March 1947.

22. SCS-TP-106 - Hydraulic Design of the Box Inlet Drop Spillway, July 1951.

23. Hydraulic Model Studies for Whiting Field Naval Air Station.

24. Fortier, S. F., and Scobey, F. C. - "Permissible Canal Velocities"; American Soc. Civ. Engin. Trans. 89, p. 940-956.

25. Lane, E. W. - "Progress Report on Results of Studies on Design of Stable Channels"; Hydraulic Laboratory Report No. Hyd-352, June 1952; U. S. Bureau of Reclamation.

26. "Standards for Permissible Non-Eroding Velocities"; Bureau of the Methodology of the Hydro-Energo Plan; Gidrotekhniches Koye Stroitel'stvo, Obedinennoe Nanchno- Tekhnicheskoe 12 dated'stvo., Moscow, USSR, May 1936.

27. Einstein, H. A. - "The Bedload Function for Sediment Transportation in Open Channel Flow"; USDA Tech Bull. No. 1026, Sept. 1950.

28. Ning Chien - Meyer-Peter Formula for Bedload Transport and Einstein Bedload Function; Univ. Cal. Inst. Eng. Research, M. R. D. Sediment Series No. 7, March 1954.

REFERENCES (Continued)

29. Shulits, Samuel - The Schoklitsch Bedload Formula; Engr. J. 139, 1935; p. 644-646, 687.

30. Colby, B. R., and Hubbell - "Simplified Methods for Computing Total Sediment Discharge with the Modified Einstein Procedure"; USGS Water Supply Paper 1593; 1961.

31. Dawdy, D. R. - "Depth-Discharge Relations of Alluvial Streams-Discontinuous Rating Curves," USGS Water Supply Paper 1498-C, 1961.

32. Horton, R. E. - "Separate Roughness Coefficients for Channel Bottom and Sides," Eng. News-Record, Vol. III, No. 22; pp. 652-653; Nov. 30, 1933.

33. King, Horace Williams and Brater, Ernest F. - Handbook of Hydraulics; McGraw-Hill Pub. Co.; 1963.

34. Taylor, Donald W. - "Fundamentals of Soil Mechanics," John Wiley & Sons.

35. Nilmar Janbu - "Stability Analysis of Slopes with Dimensionless Parameters"; Harvard Soil Mechanics Series No. 46.

36. Justin, Hinds and Creager - "Engineering for Dams"; Vol. III; John Wiley & Sons.

37. Bishop and Bjerrum - "The Relevance of the Triaxial Test to the Solution of Stability Problems", from Research Conference on Shear Strength of Cohesive Soils, University of Colorado, June 1960; pp. 437-501.

38. Bennett, P. T. - "The Effect of Blankets on Seepage Through Pervious Foundations"; ASCE Trans. Vol. III, 1946, page 215.

39. Agricultural Research Service - "Study and Investigations of Use of Materials and New Designs and Methods in Public Works"; Committee Print No. 6, 87th Congress, 2nd Session; U. S. Gov. Printing Office, Washington, D. C.; 1962.

40. Parsons, D. A. - "Vegetative Controls of Streambank Erosion."

41. Bondurant, D. C. - "Channel Rectification Structures."

REFERENCES (Continued)

42. Gildea, A. P. - "Design Practice for Levee Revetment on West Coast Intermittent Streams."

43. Department of Public Works, State of California - "Bank and Shore Protection in California Highway Practice"; Nov. 1960.

44. Soil Conservation Service - National Engineering Handbook, Section 14, Chute Spillways.

45. Soil Conservation Service - "Conservation Irrigation"; Agriculture Handbook No. 107.

46. Beauchamp, K. H. - "Structures in Soil and Water Conservation"; Soil Conservation Service, E&WP Unit, Milwaukee, Wisconsin.

47. Linder, W. M. - "Stabilization of Stream Beds with Sheet Piling and Rock Sills."

48. Vanoni, V. A. and Brooks, N. H. - "Laboratory Studies of the Roughness and Suspended Load of Alluvial Streams," U. S. Army Corps of Engineers - Missouri River Division Sediment Series No. 11, December 1957.

49. Chien, N. - "A Concept of the Regime Theory", Transactions, ASCE, Vol. 122, Paper No. 2884, 1957, pp. 785-793.

50. Blench, J. - Mobile-Bed Fluviology, University of Alberta, Dept. of Technical Services, 1966.

51. Simons, D. B. and Albertson, M. L., "Uniform Water Conveyance Channels in Alluvial Material," Transactions, ASCE, Vol. 128, Part 1, Paper No. 3399, 1963, pp. 65-107.

52. Soil Conservation Service - National Engineering Handbook, Section 3, Sedimentation.

247-927 O - 78 - 16